DEFINITE PROJECT REPORT FOR WEST PETERBORO RESERVOIR

MERRIMACK RIVER BASIN FLOOD CONTROL

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APPENDIXES

Corps of Engineers, U. S. Army

U. S. Engineer Office — Boston, Mass.

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WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
3D FLOOR, PARK SQUARE BLDG.
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BOSTON, MASS.

August 5, 1940
(Revised - December 14, 1940)

Subject: Definite Project Report for West Peterboro Reservoir

To: The Chief of Engineers, U.S. Army, Through the Division Engineer, North Atlantic Division, New York, N.Y.

1. Project Authority.- West Peterboro Reservoir is proposed as a unit of the comprehensive plan for flood control reservoirs and related flood control works for the Merrimack River Basin authorized by the Flood Control Acts approved June 22, 1936, and June 28, 1938. The Flood Control Act approved June 28, 1938, provides that, "The project for flood control in the Merrimack River Basin, as authorized by the Flood Control Act approved June 22, 1936, is modified to provide, in addition to the construction of a system of flood control reservoirs, related flood control works which may be found justified by the Chief of Engineers."

2. Previous Investigations.- The investigations and studies upon which the authorized project for the Merrimack River Basin is based are described in House Document No. 689, 75th Congress, 3d Session. A comprehensive plan of flood control reservoirs and related flood control works, at an estimated total cost of $21,000,000, was recommended in that document and authorized by the Flood Control Act approved June 28, 1938. Definite project reports have been approved for three flood control reservoirs under the general Merrimack Basin authorization, as follows:

Reservoir	Estimated Cost for Flood Control	Estimated Cost of Power Provisions Not Included in Original Estimates	Status
Franklin Falls	$ 7,488,000	$ 395,000	Construction started October 1938.
Blackwater	1,120,000	180,000	Construction started June 1940.
Hopkinton-Everett	11,300,000	-	Definite project approved March 12,1940.
Total	$19,905,000	$575,000	

3. Preliminary definite project reports on Mountain Brook Reservoir and West Peterboro Reservoir were submitted April 25, 1940, as a part of the comprehensive plan and were approved by the Chief of Engineers on April 30, 1940. A final definite project report on the Mountain Brook Reservoir was submitted on June 1, 1940. This reservoir has a drainage area of 14 square miles and a flood control storage capacity of 4,800 acre-feet, at an estimated cost of $370,000.

4. At a hearing held by the New Hampshire Water Resources Board in Concord on May 8, 1940, the Federal Power Commission proposed a plan of multiple-purpose development for the Contoocook River Basin as an alternate to the Department's plan. A review of the Commission's proposal has been prepared by the District Engineer and was submitted to the Department on August 12, 1940. West Peterboro Reservoir, as proposed herein, is common to both the Federal Power Commission's and the Department's plans. The status of this definite project, therefore, will not be affected by any developments resulting from consideration of the Commission's alternate system.

5. Definite Project Plan.-

a. Location and Description.- The proposed West Peterboro Reservoir is located in Hillsboro and Cheshire Counties, New Hampshire. The dam site is about one-half mile upstream from the village of West Peterboro, N.H., on Nubanusit Brook, a tributary of the Contoocook River, and about 35 miles southwest of Concord, N.H., and 60 miles northwest of Boston, Mass. (See Plates 1 and 2.) With spillway crest at Elev. 946, the reservoir will have an area of 900 acres, consisting of about 60 percent of wooded area, 17 percent in pasture and meadow, and 23 percent tillable land. The area affected is sparsely populated, involving fewer than 50 persons. There are no railroads in the area, and only 1.5 miles of highways and a small amount of power and telephone line relocation will be necessary.

b. Reservoir Capacity.- The reservoir is designed to control a flood of about 100-year frequency and similar in magnitude and manner of occurrence to the March 1936 flood, the maximum of record. The flood control storage capacity of 16,000 acre-feet at spillway crest Elev. 946 is equivalent to 6.8 inches of run-off over the 44-square-mile drainage area. The corresponding outlet design discharge is 650 c.f.s. with the pool at spillway lip elevation. It is planned to obtain this discharge through a single outlet controlled by two gates. With one gate open the desired design discharge is attained. For emptying purposes, both gates will be opened, permitting a maximum discharge of 1,000 c.f.s., which is about the limit of capacity of the channel downstream. Under this plan, 90 percent of the capacity of the reservoir can be emptied in 13 days. Details of the hydrology and hydraulic factors involved are given in Appendixes A and B.

c. Conservation Storage and Power Development Possibilities.- The proposed West Peterboro Reservoir site is favorably adapted to greater storage capacity than required for flood control alone. The optimum economic additional capacity is about 33,000 acre-feet. This additional amount could be obtained by raising the proposed initial spillway twenty-two feet. The required flood control storage of 16,000 acre-feet would then be available between elevations 959 and 968, the new spillway crest, and the capacity below Elev. 959, 33,000 acre-feet, would be available as conservation storage. In addition, a power installation of about 4,000 kw. would be feasible. These possibilities are described in detail in Appendix E. The development of the project for combined use is not desired at this time, but, since it appears that the multiple-purpose possibilities are justified economically, it is planned to make provision in the initial construction so that the dam may be raised and the power installation added in the future. The estimated cost of making the necessary provisions at this time for possible future conservation and power development is $100,000. -> $85,000

d. Spillway Requirements.- The spillway design flood is based on the maximum rainfall rates estimated by the Hydrometeorological Section of the U. S. Weather Bureau. (See Appendix A.) The proposed spillway is adequate to pass a flood with a volume of 18 inches and a peak inflow of 50,000 c.f.s.(1135 c.f.s. per sq.mi.). The maximum surcharge on the spillway for this flood would be 15.9 feet and the freeboard from the top of dam, 5.1 feet. Details of the spillway requirements and spillway design are given in Appendixes A and B.

6. Description of Structures.- The main retaining section of the proposed dam (see Plate 3) will consist of a rolled earth-fill embankment about 950 feet long with a maximum height of 65 feet and side slopes of 1 on 3 in general. The spillway, located on the right abutment adjacent to the embankment, will have a crest length of 100 feet at Elev. 946. The outlet works, consisting of a single conduit with a cross-sectional area of 32.3 square feet and controlled by two vertical gates, each 3'-3" x 5'-0", will be founded on bedrock of the right abutment. The structures will be designed to facilitate raising the dam 22 feet for possible future conservation storage and power development and an intake provided for a 9-foot diameter penstock. Sufficient quantities of suitable impervious and random material for the embankment are available from the structure excavation. It is planned to divert the river through the penstock during construction, which will enable placing practically all excavated material directly in the embankment. A

more complete description of the geology and soil data at the dam site and the criteria involved in the selection of the structures is contained in Appendixes C and D.

7. Estimate of Cost.- The estimated first cost of the proposed West Peterboro Reservoir is as follows (for details, see Appendix F):

Construction	$1,015,000
Lands, rights-of-way, relocations. .	155,000
Total estimated first cost	$1,170,000

8. Economic Analysis.- The economic analysis of the proposed West Peterboro Reservoir for flood control is shown in the following tabulation. Analysis of the economics of possible future development for multiple purposes is given in Appendix E.

Item	West Peterboro Reservoir*	Other Reservoirs Proposed for Comprehensive Plan**	Total all Reservoirs
Net drainage area (sq.mi.)	44	1,573.5	1,617.5
Flood control storage - (acre-feet)	16,000	380,800	396,800
(inches)	6.8	4.5	4.6
Estimated first cost	$1,170,000	$20,853,000	$22,023,000***
Annual carrying charges	$54,100	$965,800	$1,019,900
Annual benefits	$58,000	$1,085,000	$1,143,000
Ratio of benefits to costs	1.07	1.12	1.12
Cost per acre-foot of storage	$73	$55	$56
% of Merrimack Basin damages prevented	4	70	74

*Flood control project with dam designed to provide for future raising.

**Franklin Falls, Blackwater, Hopkinton-Everett and Mountain Brook.

***Includes $675,000 for power provisions not contemplated in the original estimates on which the $21,000,000 authorization was based (see paragraphs 2 and 5c).

DEFINITE PROJECT REPORT FOR WEST PETERBORO RESERVOIR

MERRIMACK RIVER BASIN FLOOD CONTROL

APPENDIX A

HYDROLOGY

1. Reference.- Reference is made to Engineer Bulletin R.& H. No. 9, 1938, subject: "Spillway Capacities," which directs, in paragraph 19, that a hydrology report will be submitted to the Office of the Chief of Engineers for approval as the first step in the final design of a project.

2. Project Description.- The West Peterboro dam site is located on Nubanusit Brook about 1/2 mile upstream from West Peterboro. (See Plate 2.) The reservoir has a tributary drainage area of 44 square miles, which is 90 percent of the drainage area of Nubanusit Brook and approximately 36 percent of the total drainage area above Peterboro, N.H. The reservoir will provide flood protection principally for lower Peterboro, and, to some extent, for the 32 miles of Contoocook River Valley between Peterboro and Hopkinton-Everett Reservoir. The flood control storage provided is 16,000 acre-feet, equivalent to 6.8 inches over the drainage area. Area and capacity curves are shown on Plate 9. The reservoir, designed initially for retarding basin operation, will have a single flood control outlet. This outlet will be provided with two gates, one of which will remain open at all times to assure the required reservoir discharge during floods, while the second will be opened after the flood to accelerate emptying the reservoir. The outlet design discharge for flood control is 650 c.f.s. with reservoir water surface at spillway crest. The initial plan provides for a 100-foot spillway at Elev. 946.

3. Basin Characteristics.- The watershed of Nubanusit Brook above the proposed West Peterboro dam site is an elongated basin with a maximum length of 11 miles and a width of 5 miles. The long axis is in a north and south direction (see Plate 2). The area is drained by two principal tributaries: first, Nubanusit Brook which rises in the northern portion and flows generally southeasterly, and second, Stanley Brook which rises in the southern section and flows northeasterly. These tributaries unite within the proposed reservoir area about one mile above the proposed dam site. Because of this characteristic of two principal tributaries rising in opposite extremities of the basin, there is less likelihood of experiencing as severe a run-off from a concentration of rainfall over the entire

basin than is possible in a fan-shaped drainage area. The drainage areas of the two tributaries are approximately equal. It is probable that flood flows from both areas will arrive in the reservoir at about the same time when produced by reasonably uniform, simultaneous rainfall over the entire watershed. The upper sections of both tributaries contain several comparatively large natural lakes. On Nubanusit Brook these are: Spoonwood Pond, Nubanusit Lake, Harrisville Pond, and Skatutakee Lake. Nubanusit Lake, the largest, has an area of about 1-1/2 square miles and a storage capacity of about 4,600 acre-feet, amounting to about 14 inches over its drainage area of 6 square miles. The water bodies on Stanley Brook are Thorndike Pond and Frost Pond, located in the southern part of its drainage area. Most of these lakes have been increased somewhat beyond their natural size by the construction of small embankments. The additional capacity obtained by these artificial means, however, is not the major portion of the capacity of the lakes, and the failure of these small embankments would not, therefore, result in the release of the entire volume of water in the lakes.* Although these lakes are used to some extent for flow regulation, they are generally maintained at a maximum level for scenic and recreational purposes and cannot be counted on, therefore, for appreciable flood control use.

4. The outer margin of the drainage area is at high elevations, the west border averaging about Elev. 1800 with a maximum of 2800, and the east border averaging about Elev. 1100 with a peak of 2000. Steep slopes on the margin of the drainage area are conducive to rapid run-off. There is some valley storage in the lower reaches of both tributaries. The profiles of the two principal tributaries are shown on Plate 10. A graph of increments of drainage area versus river miles above the proposed dam site is also given on this plate. This graph indicates that the largest contributory area is located about 8 miles above the dam site. It also shows that about 25 percent of the total drainage area beyond the 9-mile point is above Skatutakee Lake on the Nubanusit branch and that the shape and location of this area are such as to minimize its effect on the peak flow at the dam.

5. <u>Stream Flow Data</u>.- The U. S. Geological Survey maintained a gaging station on Nubanusit Brook from 1920 to 1931, with a drainage area of 48.1 square miles. The maximum peak discharge during this period was 1,130 c.f.s., recorded during the November 1927 flood. Peak values determined by the U. S. Geological Survey at a dam near West Peterboro (45.2 square miles) for the 1936 flood and the 1938 flood are 4,100 and 4,140 c.f.s., respectively. Discharge records for other drainage areas nearby or adjacent were studied for obtaining run-off characteristics. Records for the following stations were used in this study. (See Plate 11.)

*On recommendation of the Office, Chief of Engineers, an additional allowance of one foot in computed maximum water surface will be made to allow for the possible failure of these upstream embankments.

Basin	Name of Stream	Station Location	Drainage Area sq.mi.	Max. Discharge c.f.s./sq.mi.
Merrimack	Contoocook River	East Jaffrey	36.1	99
	Nubanusit Brook	West Peterboro	45.2	92
	No.Branch Contoocook River	Antrim	54.8	85
	Warner River	Bradford	19.7	120
	Souhegan River	Greenville	29.9	206
	Stony Brook	Wilton	33.2	174
Connecticut	Moss Brook	Wendell Depot	12.2	126
	So.Branch Ashuelot River	Webb	36.6	163
	Otter Brook	Keene	41.8	147

6. Precipitation Records.- Precipitation records are available from nearby stations at Greenville, Peterboro, Keene, Fitzwilliam, and Minnewawa, N.H., and Winchendon, Mass. The locations of these stations are shown on Plate 11. There are no recording rain gages nearby, hence data on intensities of precipitation of the desired accuracy for this small drainage area could not be obtained. The maximum intensity of rainfall known in this vicinity is the unofficial record for September 21, 1938, at Peterboro, N.H., when 9-1/2 inches of rainfall in two hours was reported.

Selection of Spillway Design Flood

7. Distribution Values.- An analysis of the stream flow data available on Nubanusit Brook disclosed no data suitable for determining distribution values applicable to spillway design floods. The flood of November 1927 was the largest on which adequate data were available, but the peak flow of this flood was only 1,130 c.f.s. Consideration was then given to records available on other New England streams of about 44 square miles drainage area with the object of obtaining data on the largest flood on streams with physical characteristics conducive to greater flood peak values than Nubanusit Brook. The best information obtainable was for South Branch Ashuelot River at Webb (D.A. = 36.6 sq.mi.) and Otter Brook near Keene (D.A. = 41.8 sq. mi.). Both areas are in the Connecticut River Basin adjacent to Nubanusit Brook and have recorded stage hydrographs for the 1938 flood. This flood is the largest summer flood of record in this area, comparable in peak value to the 1936 flood. Analysis of the September 1938 flood at these stations is summarized below.

Stream	Drain- age Area Sq.Mi.	Peak of Flood C.F.S.	C.F.S. Per Sq.Mi.	Volume of Run-off Inches	Rain- fall in Inches	Ave.Rate of Infil- tration Inches Per Hour	Max. % Dis- tribu- tion	Time of Lag in Hours
So.Br.Ashuelot River	36.6	5960	163	6.00	10.85	0.15	19.6	5
Otter Brook	41.8	6130	147	6.04	10.85	0.10	17.3	6

Plate 12 shows the recorded hydrograph and rainfall values with the reconstituted hydrograph for the South Branch Ashuelot River, and Plate 13 gives corresponding data for Otter Brook. The computed distribution graphs at these two stations, given on Plate 14, show a close similarity. An analysis was also made of the November 1927 flood at these two stations. Because of the lower peak value and rainfall intensity of the 1927 flood, the variations resulting from the derivation of 3-hour rainfall values from 24-hour observations made it impossible to define distribution values comparable to those obtained from the 1938 flood. Studies of distribution values for small drainage areas with short time increments showed it was most essential to have accurate rainfall intensity data, because small changes in the time distribution result in material changes in the distribution values.

8. The South Branch Ashuelot River and Nubanusit Brook are both two-branch streams with similar profile slopes and general physical characteristics. Comparing areal distribution in relation to the point of outflow on the curve, Increments of Drainage Area vs. River Miles (Plate 10), it will be noted that the maximum concentration of area in both streams lies about at mile 8. The shape of the South Branch Ashuelot River curve definitely shows this stream to be conducive to higher flood peak values considering only the areal distribution to mile 9.5. Inspection of the stream pattern shows, for the main stream above Troy, a marked fan shape, productive of extreme concentration of flow for at least one-third of the drainage area; also, valley storage is negligible and much less than available on Nubanusit Brook. Furthermore, the areal distribution on Nubanusit Brook beyond mile 9.5, amounting to about 25 percent of the total area, is such as to minimize its contribution to peak flows. Applying the distribution values obtained on the 36.6 square miles of drainage area of the South Branch Ashuelot River to the 44 square miles of drainage area of Nubanusit Brook is conservative for floods of the 1938 magnitude. This is also true, to a somewhat lesser extent, of the Otter Brook distribution values.

9. Application of these distribution values, based, in the case of the South Branch Ashuelot River, on a peak flow of 163 c.f.s. per square mile, to a run-off of spillway design magnitude is subject to question. It was therefore decided to increase the 19.6 percent maximum distribution value obtained from the South Branch Ashuelot River to 25 percent, an increase of about 28 percent, adjusting the distribution graph accordingly. This value was in line with those obtained by application of Snyder's method to both the South Branch Ashuelot River and Nubanusit Brook, although the latter stream, because of the location of its two principal tributaries, required break-down into two separate areas. After consideration of all factors discussed above, the distribution graph shown on Plate 14 was selected for the spillway design for West Peterboro Dam. The 3-hour unit hydrograph based on these adopted distribution values is shown on Plate 15. This hydrograph has a peak of 2,500 c.f.s., or 57 c.f.s. per square mile for a one-inch run-off in 3 hours.

10. Maximum Storm.- Studies were made of both the summer-fall and winter-spring limiting storm conditions. The high intensity summer-fall rainfall values shown in Plate 16 proved the more severe criteria for spillway design. This intensity curve is based on limiting precipitation values determined by the Hydro-Meteorological Section of the U. S. Weather Bureau and has been used to obtain satisfactory precipitation data for the hydrology analysis of Blackwater Reservoir and the proposed Hopkinton-Everett Reservoir. The values used for the proposed West Peterboro Reservoir were substantiated by a recent report on the "Maximum Possible Precipitation over the Ompompanoosuc Basin above Union Village, Vermont," prepared by the Hydro-Meteorological Section of the Weather Bureau (March 18, 1940). On Plate 17, which is a reproduction of "Enveloping Duration-Depth Curves of Maximum Possible Rainfall over Selected Basins in the New England Region," as contained in that report, is shown the duration-depth curve used for West Peterboro Reservoir, with values taken from Plate 16 up to 24 hours and with the 30-hour value made comparable to the similar values on Plate 17. The duration-depth curve used for Mountain Brook Reservoir (D.A. = 14 square miles) is also illustrated. (Definite Project Report for Mountain Brook Reservoir, submitted June 1, 1940.) The data concerning the winter storm with snow run-off conditions, as discussed in this Weather Bureau report, were utilized to extrapolate run-off values for the spillway design flood. The spillway requirements were practically identical with those for the summer flood, but because of the extrapolations required in the winter run-off, the summer flood was accepted as the more reasonable one. Computations and results of these two floods are outlined in the following paragraph.

11. <u>Computed Spillway Flood</u>.- Spillway floods were computed, using distribution values shown on Plate 15, for both summer-fall and winter-spring precipitation conditions. An infiltration rate of 0.05 inch per hour and base flow of 5 c.f.s. per square mile were used. The assumed minimum infiltration rate of 0.05 inch per hour, or 0.15 inch per 3-hour period was based on the smallest rate determined from analyzing floods of record for unit hydrographs in this area. The computations for the summer-fall flood are tabulated on Plate 18. The hydrograph and pluviagraph are shown on Plate 19. The winter-spring flood was based on an extrapolation of data summarized on Table VII and Figure 41 of the report on the Ompompanoosuc Basin. These references tabulate and show graphically the area-depth relation of maximum possible rainfall plus snow melt over selected basins in the New England region. The hydrograph and pluviagraph for the winter-spring flood are shown on Plate 20.

12. <u>Reservoir Operation Assumptions</u>.- It is assumed that the reservoir will be filled to normal maximum pool elevation of 946 feet M.S.L., the crest of the spillway, at the beginning of the spillway design flood. The outlets are assumed to be inoperative.

13. <u>Spillway Rating Curve</u>.- The spillway rating curve (Plate 23) was computed for a free overfall Ogee spillway using the weir formula $Q = CLH^{3/2}$, where "L" = 100 feet and values of "C" of an Ogee section were taken from 3.0 to 3.8 for the maximum head.

14. <u>Method of Routing</u>.- The unit graph used applies to reservoir inflow, consequently the spillway floods were routed through the reservoir using the gross surcharge storage.

15. <u>Effect of Computed Spillway Floods</u>.- The two computed spillway floods, based on summer-fall and winter-spring conditions, were routed through the reservoir to obtain reservoir discharge and the maximum water surface elevation. Although the peak inflow of the summer flood was 33,200 c.f.s. compared with 26,000 c.f.s. for the winter storm, the peak outflows were almost identical: 19,200 c.f.s. for the summer flood and 19,000 c.f.s. for the winter flood. This discharge similarity is caused by the different volume distribution in the two floods; the summer storm having a short period of intense rainfall producing the high peak inflow, with the winter storm having greater volume but with less intensity. The data pertaining to these two computed spillway floods are summarized as follows:

	Computed Summer-Fall Flood	Computed Winter-Spring Flood
Duration of storm, hours	24	30
Rainfall, inches	19.20	12.60
Snow melt, inches.	none	10.80
Total, inches	19.20	23.40
Rate of infiltration (inches per hour)	0.05	0.05
Run-off, inches	18.0	21.90
Volume, acre-feet	42,200	51,400
Volume, acre-feet (including base flow)	43,620	52,900
Peak inflow, c.f.s.	33,200	26,000
Peak outflow, c.f.s.	19,200	19,000
Maximum pool elevation, M.S.L.	959.7	959.6
Surcharge storage, inches	7.5	7.5
Freeboard, feet.	5.3	5.4

The inflow ordinates of the computed spillway floods were then increased 25, 50, and 80 percent and similarly routed through the reservoir. The data derived from the three routings of the summer-fall flood are shown graphically on Plate 21, where the percent of the computed spillway flood is plotted, (a) against the pool elevation in feet above mean sea level, and (b) against peak inflow and outflow.

16. Possible Variation in Shape and Peak of Computed Spillway Flood.- Of all factors entering into the selection of the spillway design flood, the most uncertain are the shape and peak of the hydrograph. These depend on distribution values selected from actual flood experiences which are generally much less severe than the spillway design floods finally selected. In order to ascertain the change in maximum water surface elevation that would result from variation in the shape and peak of the computed spillway flood hydrograph of constant volume, the following analysis was made.*

17. A unit graph derived from floods of record is applicable only to rates of rainfall comparable to those used in its derivation. In the case of the base unit graph used in this report, the maximum precipitation value for a 3-hour period in the 1938 flood, from which it was derived, was 1.9 inches; hence, in the computed spillway flood the two maximum rainfall periods of 9.5 and 3.0 inches may be

*The analysis was based on the procedure followed in a report by the Office, Chief of Engineers, on spillway design for Delaware Reservoir.

considered as excessive for the use of the unit graph. A hydrograph, hereafter referred to as the "Basic Hydrograph", was first constructed from the base unit graph using all the 3-hour periods of rainfall except the 9.5 and 3.0-inch periods. (See Plate 22.) Then the unit graph was applied to all periods of rainfall to obtain hydrograph "A", which corresponds to the computed spillway flood for summer-fall conditions as described in paragraph 11. Three variations of this flood were then obtained by increasing the peak of the unit graph as shown in the following table. Hydrograph "B" was obtained by increasing the peak of the unit graph by 32 percent and shortening the time of lag by approximately one hour and applying this revised unit graph to the two maximum 3-hour rainfall values. This partial hydrograph was then added to the "Basic Hydrograph" in the correct time relationship resulting in hydrograph "B". The peak of this flood was 38,200 c.f.s., or 15 percent in excess of the computed spillway flood (hydrograph "A"). Similarly, hydrographs "C" and "D" were computed by further peaking up the base unit graph and applying it to the two maximum periods of rainfall. These three additional hydrographs ("B", "C", and "D") were routed through the reservoir to determine the outflow peak and the maximum water surface elevation. The data obtained from these routings are tabulated in the following table and are shown graphically on Plates 21 and 22.

Item	Hydrographs: A	B	C	D
Volume of run-off, inches	18.0	18.0	18.0	18.0
% Increase, unit graph peak	100	132	168	200
Inflow peak, c.f.s.	33,200	38,200	44,000	50,000
C.F.S. per square mile	755	869	1,000	1,138
% of basic inflow peak	100	115	132	151
Outflow peak, c.f.s.	19,200	20,500	21,300	21,750
% of basic outflow peak	100	107	112	114
Max. water surface elevation	959.7	960.3	960.7	960.9

18. Discussion of the Factors Involved in the Selection of the Spillway Design Flood.- a. The maximum rainfall values used were adjusted to conform to the latest data available in New England as compiled by the U. S. Weather Bureau in the report on "The Maximum Possible Precipitation over the Ompompanoosuc River Basin above Union Village, Vermont," dated March 18, 1940. The point rainfall values used as shown on Plate 16 equal or exceed for all intensities the values given in Figure 24 of the Ompompanoosuc report for maximum recorded rainfall in the United States. Likewise,

the values used for 100 square miles equal similar values given in the Ompompanoosuc report for which reliability factors were applied to obtain maximum possible rainfall. Accordingly, a factor of safety for increasing these values does not appear justified. Interpolation for the 44-square-mile drainage area of Nubanusit Brook between the point and 100-square-mile values is believed to be conservative. Therefore, no factor of safety is considered necessary to increase the rainfall values used in the computed spillway flood.

b. The possibility of the adopted maximum rainfall values being greatly exceeded on this small drainage area is considered too remote to justify a factor of safety as discussed in a. above. The effect of areal distribution on increasing the flood peak value used for Nubanusit Brook is also believed negligible, due to the shape and distribution of the drainage area. It would be impossible for a small areal concentration of rainfall to cover the steep headwater areas of both major tributaries lying at opposite ends of the basin. Basing the distribution values on those of South Branch Ashuelot River minimizes the possibility of higher flood peaks from a more critical distribution of rainfall than the average distribution used. No factor of safety is considered necessary for these items.

c. The average infiltration rate of 0.05 inch per hour, or 0.15 inch per 3-hour period, is low enough to eliminate the need of a factor of safety for the assumed run-off conditions.

d. The unit graph and distribution values selected for the computed spillway flood, as shown on Plate 15, give a peak value of 2,500 c.f.s., or 57 c.f.s. per square mile for one inch of run-off in 3 hours. A comparison of these values with similar data in the Office, Chief of Engineers for other drainage basins of comparable size indicated that the peak of the unit hydrograph might well be increased to 80 or 90 c.f.s. per square mile as a factor of safety to allow for the possibilities of error in extrapolating the values from small known floods to a flood of spillway design magnitude.

e. The spillway is a free overfall Ogee concrete weir, the capacity of which can be accurately determined. No factor of safety is required on this account.

f. The theoretical freeboard required for West Peterboro Reservoir, based on the criteria outlined in Engineer Bulletin R. & H. No. 9, 1938, is 4.8 feet. The minimum freeboard of 5 feet was adopted, therefore, for the requirements for wave action. The storage capacity afforded at the range of stage represented by this freeboard amounts to 8,800 acre-feet, or about 3.7 inches over the drainage area controlled. In addition, on recommendation of the

Office, Chief of Engineers, an allowance of about one foot increase in the maximum water surface will be made to allow for the possibility of failure of small embankments upstream from the proposed West Peterboro Reservoir (see paragraph 3). The storage available in this additional one-foot allowance amounts to 2,000 acre-feet, or about 0.9 inch over the drainage area controlled. These amounts of storage above the maximum computed water surface and top of the dam are appreciable with respect to the total volume of the spillway design flood, and no further allowance is considered necessary as a factor of safety for this item.

19. Selected Spillway Design Flood.- After analysis of the foregoing factors, it was originally considered that the computed spillway flood as shown on Plate 19 would be adequate for a spillway design flood for this reservoir. At the request of the Office, Chief of Engineers, however, further consideration was given to the use of a higher peaked unit hydrograph. After such further consideration, a unit hydrograph was adopted with a peak 100 percent greater than that of the unit graph originally derived. This resulted in a spillway design flood equivalent to hydrograph "D" on Plate 22. The adopted unit hydrograph and the inflow-outflow stage hydrographs for the spillway design flood are shown on Plates 29 and 30. The adopted spillway design flood has a peak inflow of 50,000 cubic feet per second, equivalent to 1,138 c.f.s. per square mile of drainage area controlled, and a volume of 18 inches. The maximum water surface resulting from this flood has been computed to be Elev. 960.9. An additional allowance of one foot in the possible maximum water surface has been made to allow for the possibility of failure of upstream embankments. The top of dam has been placed, therefore, at Elev. 967, thus providing a freeboard of 5.1 feet.

20. Reservoir Design Flood.- The reservoir design flood for the West Peterboro Reservoir was based on the same criteria used for other reservoirs in this district. The storage capacity and the outlet size were selected to control a flood with approximately 100-year frequency, without spillway discharge. On this basis it was found that a storage capacity of 16,000 acre-feet would be sufficient to store the entire 100-year flood with a maximum outlet discharge (at spillway lip Elev. 946) of 650 c.f.s. As a further criterion for reservoir design it was considered desirable to test the adequacy of the selected storage volume and design outlet discharge for the maximum flood of record. The March 1936 flood on the Merrimack Basin in most cases exceeded in peak value, and definitely in volume, any flood of record. Since volume is of primary importance, reservoir design in the Merrimack Basin must recognize the spring flood as the governing criterion. The storage provided in West Peterboro Reservoir was found to be ample to control the 1936 flood. Although a spillway discharge of 600 c.f.s. occurs, it is sufficiently delayed not to affect the peak reductions at damage centers downstream. The effect on the 1936 flood at the reservoir site is shown on Plate 25.

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MERRIMACK RIVER BASIN FLOOD CONTROL

APPENDIX B

HYDRAULIC DESIGN

1. Outlets.- In the design of the flood control outlets for West Peterboro Reservoir, it was necessary to provide for two stages of development: (1) for flood control only with spillway crest at Elev. 946, and (2) for a possible future multiple-purpose development with the flood control storage allocation between Elev. 959 and a raised spillway crest at Elev. 968. Consequently, the following items were considered as design criteria for the conduit and gate capacities:

a. To discharge the design outflow of 650 c.f.s. (see paragraph 20, Appendix A) with pool at initial spillway crest Elev. 946 and tailwater at about Elev. 910 (controlled by the existing dam downstream).

b. To discharge a maximum of about 1,000 c.f.s. (maximum channel capacity) for emptying purposes with approximately same headwater and tailwater elevations as given in a.

c. In the ultimate development to discharge for flood control approximately 1,300 acre-feet per day, adjusting the flood control outlet discharge to the power discharge, maintaining in so far as possible a maximum combined rate of 650 c.f.s. during the flood. From present estimates, it appears that the discharge from full operation of the turbines will give the required discharge during the flood.

d. To discharge, for emptying purposes in the ultimate development, about 2,000 acre-feet per day with a maximum rate of 1,200 c.f.s.

2. To meet the criteria in a. and b. above, the capacity of the conduit with both gates open will be 1,000 c.f.s. with full pool and tailwater at 910. This requires a conduit area of 32.5 square feet for its length of approximately 400 feet and is obtained by a rectangular tunnel 5 feet wide and 6-1/2 feet high. Two gates will be provided, each 3'-3" x 5'-0". The design discharge of 650 c.f.s. at full pool capacity is attained with one

gate open. (See Plate 26.) With both gates open, the discharge at full pool is limited to 1,000 c.f.s., the capacity of the conduit. In the ultimate development it was not found desirable to reduce the gate sizes of the initial development, because of the transition required. Also, consideration of criteria c. and d. above made it appear unwarranted to make the gates of two different sizes, such that the larger would give the required 650 c.f.s. in the initial stage and the smaller the same discharge in the ultimate stage. This would have an additional objective in preventing inter-change of the gates in case the required gates became inoperative. Since the ultimate multiple-purpose reservoir will require continuous regulation of the power storage and, during floods, adjustment of the flood control discharge to the turbine discharge to prevent a combined outflow materially in excess of the basic design, the criteria of a fixed outlet discharge is not practicable in a conduit outlet and the required regulation can only be obtained by part gate openings. Recent data have indicated the feasibility of part gate openings within the limits required by this project. However, if it is necessary to open one of the gates proposed for the initial development during the flood, the outlet discharge will be about 800 c.f.s. and will not appreciably reduce the reservoir effectiveness. On the other hand, if the flood control gates are closed and reservoir discharge is limited to the full turbine discharge, the outflow will be about 500 c.f.s. The above considerations led to the retention in the ultimate project of the two equal size gates proposed for the initial project.

3. Spillway.- The spillway for West Peterboro Reservoir will be located in the right abutment and will consist of a short approach channel, a straight Ogee free overfall concrete weir 100 feet long, and concrete-lined discharge chute provided with a stilling basin. The approach channel floor will be at Elev. 941 (5 feet below the spillway crest), widening from 100 feet at the spillway to over 200 feet at the upstream end. The discharge capacity of the spillway (Plate 23) is based on the weir formula, $Q = CLH^{3/2}$, with values of "C" varying from 3.0 to 3.8 for maximum head. The spillway discharge channel is located with reference to the rock contours and provides a slope of approximately 11 percent from spillway weir toe to start of stilling basin floor and converges from 100 feet at spillway weir crest to 75 feet in width. The channel capacity throughout is more than sufficient to prevent backwater at the weir and to permit flow at greater than critical velocity. Computation of the depth of flow in the converging portion of the channel indicated that the height of the side walls provided 5 to 6 feet of freeboard against waves set up by convergence of the channel. This was found adequate to prevent overtopping the spillway channel walls on the basis of the more severe conditions of convergence tested in the hydraulic model of the initially proposed Mountain Brook spillway.

4. The design of the stilling basin required consideration of the following tailwater conditions: (1) with the existing dam downstream left intact during the initial development, and (2) with the dam removed in the ultimate development. Furthermore, it is not expected that the existing dam downstream could safely pass the spillway design discharge and hence stilling basin action will have to be obtained for a wide range of tailwater conditions. To allow for this uncertain tailwater, it was found desirable, both from the standpoint of economy of construction and assurance of operation, to design the stilling basin to give the required depth for the hydraulic jump under maximum conditions regardless of the tailwater downstream. This was obtained by setting the end sill crest at Elev. 900, permitting raising the stilling basin floor to Elev. 885,and the top of the stilling basin wall at Elev. 920. The latter will not be overtopped for any spillway flood that would not wash out the existing downstream dam and is amply high for conditions with the downstream dam out.

5. Tailwater Rating Curve.- The tailwater rating curve (Plate 27) is based on a computed rating of the existing concrete dam just below the proposed flood control dam. The spillway of this dam is at Elev. 909.60 and has a length of 92.6 feet. The concrete abutments at Elev. 913.65 have a combined length of 166 feet. The upper end of the rating curve on Plate 27 is based on the assumption that the dam will remain in place during the entire range of possible discharge and serves only to limit the upper possible tailwater elevation.

6. Time of Emptying.- The total time necessary to empty the reservoir with the pool initially at Elev. 946 (spillway crest) and a constant inflow of 5 c.f.s. per square mile (220 c.f.s.) is 17 days. (See Plate 28.) Both gates are assumed to be open, with a total discharge capacity of 1,000 c.f.s. with reservoir at Elev. 946. It should be noted that 4 inches, or 60 percent of the total storage, will be available in 7 days, and 6 inches, or nearly 90 percent of the total storage, will be available in 13 days. It is considered that this is adequate protection against a possible second flood.

7. Effect of West Peterboro Reservoir.- The effect of West Peterboro Reservoir on the March 1936 flood, which is the severest test flood of record, is shown on Plate 25. The inflow hydrograph is based on the flood records of adjacent tributaries, with the U. S. Geological Survey computed estimate for the peak flow at the West Peterboro Dam. The maximum reservoir outflow during this flood would be 1,250 c.f.s. comprising 650 c.f.s. from the outlet and 600 c.f.s. over the spillway. The maximum pool elevation would be 947.5. The effective reduction of the 1936 flood at downstream

damage centers is reflected in the discharge reduction on March 19th, and on this date the reservoir would be discharging the design discharge of 650 c.f.s. This would result in a reduction on the Nubanusit Brook through the town of Peterboro from 4,500 c.f.s. to about 900 c.f.s. The flood on the Contoocook River in lower Peterboro (without the proposed Mountain Brook Reservoir) will be reduced from 9,000 c.f.s. to approximately 6,450 c.f.s. Diminishing reductions will be realized downstream in Bennington and Hillsboro. Similar reductions would be obtained for the 1938 flood, for although the peak flows were slightly higher than the 1936 flood, it was a single-peak flood and the proposed reservoir could control the 1938 flood without spillway discharge. The 1938 flow in lower Peterboro would be reduced from 10,400 c.f.s. to about 7,300 c.f.s.

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APPENDIX C

GEOLOGY AND SOIL DATA

1. Description of Dam Site.- The West Peterboro dam site is located approximately 1/2 mile north of West Peterboro, New Hampshire, and approximately 1,000 feet upstream from the existing concrete dam across Nubanusit Brook. (See Plates 2 and 4.) At the site, the valley is constricted by hills which rise abruptly from the edge of the stream to give the region moderate relief. The rock at the site slopes generally eastward so that the present channel of Nubanusit Brook is located high on the west wall of the preglacial valley. The hills forming the abutments are composed largely of glacial till overlying variable deposits of glacial outwash and disintegrated mica schist. The valley bottom at the site is at present occupied by water impounded behind the existing concrete dam and has been filled with glacial and aqueo-glacial deposits underlying recent river debris.

2. Exploration at Dam Site.- The exploration of the foundation for the dam and appurtenant structures included the preliminary determination of bedrock elevations by 8 seismic lines and the drilling of 21 holes. The bedrock at drill holes D-14, D-19 to D-22, inclusive, were pressure tested to a depth of 40 feet. The location and extent of the exploration are shown on Plate 4.

3. Overburden and Rock Conditions.- The classification of the overburden and bedrock at each drill hole in the exploration is shown on Plate 5. The overburden in the left abutment is a very compact, impervious, silty glacial till which extends nearly to bedrock. A thin stratum of relatively pervious, variable silty sand and gravel immediately overlies the bedrock under the left abutment, and between this stratum and the overlying impervious till a deposit of laminated clayey sand and silt occurs having a thickness of 10 to 20 feet. The overburden in the stream bed is generally from 15 to 35 feet in depth and consists of recent, river-washed, variable sand and gravel underlain by laminated, variable clayey sand and silt with a thin deposit of variable sand and gravel resting on the bedrock. The latter two materials are extensions of the same strata found under the left abutment. The silty sand and gravel stratum is at the surface in the vicinity of D-14 and D-23. The general geologic conditions described in the foregoing are illustrated on the profile through the embankment

centerline shown on Plate 6. Typical grain-size curves of the material, except for the variable silty sand and gravel adjacent to the rock surface, are shown in Plate 7. It is estimated that the coefficient of permeability of the compact till and the laminated clayey sand and silt is about $.0001x10^{-4}$ cm/sec, and that for the variable sand and gravel stratum, $1x10^{-4}$ cm/sec. It is believed that the till possesses ample shearing strength for any earth embankment design. The shearing strength of the laminated clayey sand and silt is probably sufficient for a normal design.

4. The overburden in the right abutment is in general from 10 to 45 feet in depth and is composed of variable material. In general, a thick deposit of compact, silty till is found overlying deposits of laminated, clayey, fine sand and silt and deposits of variable silty sand and gravel. Upstream from the proposed centerline, in the vicinity of D-22 and D-23, the compact till overlies soft, disintegrated mica schist and sand which rests on bedrock and which varies in thickness from 5 to nearly 20 feet. Typical grain-size curves other than for the disintegrated schist are shown on Plate 8. It is estimated that the coefficient of permeability of the compact, silty till is $.001x10^{-4}$ cm/sec, and for the variable silty and gravelly sand and disintegrated schist, from $.001x10^{-4}$ to $10x10^{-4}$ cm/sec.

5. Bedrock is not accessible in the left abutment but is at a depth from 10 to 45 feet in the right abutment as shown by the rock contours on Plate 4. The bedrock in general is a variable soft schist which is considerably fractured and weathered along numerous seams. Pressure pumping tests in five of the drill holes located on the right abutment, under pressures varying from 30 to 45 lbs. per sq. inch and for depths of about 40 feet, showed losses ranging from .06 to 2.6 gallons per minute.

6. <u>Borrow Materials</u>.- Exploration to locate suitable materials for the dam embankment, besides the drilling in the structure excavation areas, consisted of excavation and sampling of 29 test pits and examination of all exposed faces of natural overburden in the vicinity.

7. Suitable impervious and random materials are available from the excavations for the spillway approach and discharge channels. Sand containing a small percentage of fine gravel is available for fine filter and shell material from a deposit about 3/4 mile from the site. Rock for the outer sections of the embankment is available from the structure excavations. Investigations to develop deposits of sand and gravel which are both suitable and sufficient for the production of concrete coarse aggregates and gravel for filters are not complete, but it is believed that such deposits can be located

within 7 miles of the site. If necessary, these materials can be obtained from commercially developed sources near Keene, New Hampshire, approximately 21 miles from the dam site.

a. Impervious Material.- The impervious material available from the structure excavations is a well-graded, very silty sand and gravel glacial till. The material in general is about 20 feet deep, overlying variable deposits of sand and silt which will be used as random material. Typical grain-size curves of samples are shown on Plate 8. It is estimated that the material when compacted will have a coefficient of permeability from 0.001×10^{-4} to $.01 \times 10^{-4}$ cm/sec. and an angle of internal friction of 28 degrees with 0.20 ton per square foot cohesion.

b. Random Material.- The random material from the spillway excavation is variable, ranging from uniform sandy silt to slightly gravelly sand and soft, disintegrated schist. This material, except the soft, disintegrated schist which will be wasted, will be used in the random fill section of the dam. The material in general underlies the well-graded impervious material. Typical grain-size curves of samples taken from the deposit are shown on Plate 8. It is estimated that the material when placed in the embankment will have a coefficient of permeability varying from 0.01×10^{-4} to 10×10^{-4} cm/sec. and an angle of internal friction of 36 degrees.

c. Sand for Filters and Shell.- The available material for sand filters and shell is a slightly gravelly fine to coarse sand. The nearest deposits of this material are located within a hauling distance of 3/4 of a mile from the dam site.

d. The rock for the outer embankment sections available from structure excavation is a soft, badly fractured schist.

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APPENDIX D

SELECTION OF STRUCTURES

1. Selection of Type and Arrangement of Structures.- As shown on Plate 4, the entire reach of the narrow portion of the Nubanusit Brook valley extending from the existing dam to a section approximately 1,700 feet upstream has been explored to a greater or less extent to develop the most economical location for the construction of the West Peterboro Dam. Lay-outs and estimates have been prepared of dams employing an arrangement of structures similar to that adopted and shown herein at several locations in this reach. In addition, estimates have been made of alternate types of dams, viz., a concrete dam, an earth dam with a side channel spillway, and an earth dam with the spillway cut through a remote saddle on the reservoir rim. The type and location of dam selected are the most economical and are believed to be the best adapted to the geological conditions of the site. Further, the selected location of the dam avoids interference with the existing industrial development on the stream.

2. Description of Initial Structures.- The main retaining section of the initial dam will be an earth embankment having a crest length of 950 feet and maximum height of 65 feet. (See Plate 3.) The spillway will be located on the right abutment adjacent to the embankment and separated therefrom by a concrete retaining wall designed for future raising and founded on bedrock throughout. Approach and discharge channels will be cut through earth and rock. The floor of the discharge channel lies entirely within rock and will be paved with concrete from the weir to the end of the stilling basin and suitable side walls or lining sections of concrete placed against the rock will be provided for the channel within the same limits. The weir will have a crest length of 100 feet and a design surcharge of about 16 feet. The single flood control conduit and an oversized conduit which will provide ample room for the future installation of a 9-foot diameter steel pressure penstock for development of power at the site will be located through the earth embankment and will be founded on bedrock of the right abutment. A common intake structure for these conduits will be located at the upstream toe of the dam which will house two gates, each 3'-3" x 5'-0", for flood control regulation and, in the future, one gate, 6'-6" x 10'-0", for the power penstock.

3. The economics of the arrangement of structures is apparent in the fact that materials excavated from the spillway are suitable and sufficient for fill in the impervious and random fill sections of the embankment. Rock from structure excavation is available in sufficient quantities to provide ample protection of the upstream slope and the downstream toe of the embankment and the river banks in the vicinity of the spillway discharge channel. It is planned to divert the river through the penstock instead of the flood control outlets in order to utilize the larger area of the penstock and to enable placing the major portion of materials from structure excavation directly in the embankment. This plan will require completion of the conduits and excavation of a channel from the end of the power conduit to the river prior to cofferdam construction in the river and excavation for the spillway.

4. Description of Possible Ultimate Construction.- The ultimate multiple-purpose development of the reservoir to a spillway elevation of 968 will require alterations in the dam consisting principally of the construction of a concrete weir in the spillway channel 22 feet above the initial weir crest, raising the embankment retaining wall varying amounts up to 19 feet, for which provision will be made in the initial construction by the installation of a wide base wall, and raising and widening the earth embankment. Other alterations involve raising the intake structure and reconstruction of the service bridge from this structure to the top of the embankment. For development of power at the site, completion of the penstock provisions will be required consisting of the installation of the penstock, head-gate, and trash-racks.

5. Preliminary Design of Initial Embankment.- Based on the preliminary investigations of borrow materials and the embankment foundation, as described in the foregoing paragraphs, a tentative design of the initial embankment for the dam has been prepared as shown on Plate 3. The embankment will be provided with impervious and random sections, upstream rock blanket, and drainage features as described in the following paragraphs:

a. Impervious Features.- The main body of the embankment will consist of compacted impervious fill selected from the spillway excavation and corresponding to the material described in paragraph 7a. of Appendix C. A cut-off trench to bedrock will be provided in the right abutment and in a portion of the stream bed to prevent excess seepage in these regions through deposits of silty sand and gravel. Bedrock exposed in the excavation of the cut-off trench will be pressure grouted.

b. Random Section.- The downstream section of the embankment between the impervious fill and the shell will consist of random material described in paragraph 7b. of Appendix C. This material is slightly more pervious than the impervious fill material but is extremely variable.

c. Pervious Features.- Upstream and downstream shell sections of slightly gravelly sand will flank the impervious and random fill portions of the embankment. The upstream shell will be provided further with a gravel layer separating the sand section from the outer rock fill.

d. Foundation Drainage Features.- To control the seepage which will pass through the silty sand and gravel layer next to the rock surface beyond the limit where it is practical to construct a cut-off to the rock surface, a drainage trench will be excavated into this relatively pervious stratum and backfilled with sand and screened gravel. As an additional precaution to insure against piping, drainage wells will be provided under the downstream section which tap the underlying relatively pervious layer of silty sand and gravel.

e. Slope Protection.- The upstream slope of the dam will be protected with rock fill. The downstream slope will be provided with a sod surface, except that the toe of the slope will be protected with rock fill against possible eddies from a spillway discharge.

6. Ultimate Embankment Design.- The general design of the embankment for the ultimate development of the reservoir is shown in outline form on Plate 3. The principal materials required for enlarging the embankment to an increased height of 19 feet will be obtained from borrow pits located within 3/4 of a mile of the site and will consist of well-graded, very silty sand and gravel, glacial till for extension of the impervious section, and slightly gravelly sand for the downstream portion of the embankment. These materials correspond to those described in paragraphs 7a. and 7c. of Appendix C, respectively. The drainage features installed in the initial construction will be made to serve for the ultimate development by constructing drainage outlets to the extended toe of the embankment.

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APPENDIX E

POSSIBILITIES FOR CONSERVATION STORAGE AND POWER DEVELOPMENT

1. Studies have been undertaken to determine the feasibility of obtaining storage capacity over that required for flood control and for possible development of the 166 feet of gross power head available at the West Peterboro dam site. These studies indicate that 33,000 acre-feet is the optimum additional capacity. This additional amount can be obtained by raising the dam and spillway 22 feet. The requisite flood control storage capacity of 16,000 acre-feet (or 6.8 inches) can be provided between elevations 959 and 968, the new spillway crest. The storage below Elev. 959, 33,000 acre-feet, can then be made available in the interests of power and conservation (see Plate 9).

2. Pertinent data concerning the possible multiple-purpose development of West Peterboro are summarized below:

Spillway crest elevation	968 feet above M.S.L.	
Maximum power pool elevation	959 " " "	
Tailwater elevation	793 " " "	
Total storage capacity	49,000 acre-feet	
Flood control storage capacity	16,000 " "	
Power storage capacity	27,000 " "	
Dead storage capacity	6,000 " "	
Length of Penstock - - - -	4,000 feet 9' dia	10' head loss
Regulated minimum flow	56 c.f.s.	380 C.f.s.
Maximum gross head	166 feet	156 ft.
Capacity of power installation	4,000 kw.	4000 K.W.
		14.7% L.F.

3. Estimated Cost.- Development of the reservoir to the possible second stage outlined above would involve the acquisition of an additional reservoir area of 1,040 acres, including the land and buildings of the summer camp of the Sargent School of Physical Education of Boston University. This camp is an important educational development of considerable value. The relocation of an additional 5.6 miles of highway would also be required. In addition, it would be necessary to acquire the water rights of the existing 1,000-horsepower industrial installation just below the proposed West Peterboro site. The principal items of construction

involved are raising the spillway weir, the embankment wall and the embankment 22 feet, raising the control shaft of the outlet, construction of a bridge from the embankment to the control shaft, reducing the size of the flood control gates, raising the penstock intake structure, construction of a bridge to the intake, and the installation of gates, steel pressure penstock and power house. The estimated cost of the possible second-stage development, exclusive of the first-stage costs for flood control (see Appendix F) is as follows:

Dam and reservoir	$ 676,000
Lands and rights-of-way	589,000
Power installation	567,000
Total additional cost for power development . .	$1,832,000

The annual carrying charges on the above costs are estimated at $95,000.

4. Prospective Benefits of Multiple-Purpose Development.- The existing power developments below West Peterboro, totalling 375 feet of developed head, will benefit from the operation of the West Peterboro storage by increased low-water flow to the extent of 3,900,000 kw-hrs. annually. There are potential power sites at Long Falls and Penacook on the Contoocook which also would be enhanced by the operation of the power storage at West Peterboro, but no allowance has been made for these possible benefits in this analysis. The dependable capacity of the power development at the West Peterboro site would be 4,000 kw., giving an average annual energy output of 6,400,000 kw-hrs. Using power values formulated by the Federal Power Commission, namely $17.50 per kw. of dependable capacity and 2 mills per kw-hr. of energy output, the annual power benefits are $82,800 at the site and $28,800 at the existing downstream developments on the Contoocook and Merrimack Rivers computed as follows:

a. Benefits at the site:

Dependable capacity, 4,000 kw. at $17.50 . .	$ 70,000
Average annual energy output - 6,400,000 kw-hrs. at 2 mills	12,800

b. Benefits to existing power developments below reservoir:

Increase in prime peaking capacity - 1,200 kw. at $17.50	21,000
Increase in average annual output - 3,900,000 kw-hrs. at 2 mills	7,800
Total power benefits	$111,600

5. Conclusions.- The prospective power benefits that would result from a second-stage development of West Peterboro Reservoir for conservation storage and power purposes would exceed the additional costs, giving a ratio of benefits to costs of 1.17. It is concluded that this prospective benefit is sufficient to warrant making provision in the initial construction to facilitate the ultimate development of the reservoir for multiple-purpose use.

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APPENDIX F

DETAILED ESTIMATE OF COSTS

1. The estimated costs of the proposed West Peterboro Reservoir for flood control purposes (spillway lip Elev. 946) with provisions for possible future conservation storage and power development are as follows:

I. RESERVOIR COSTS

	Quantity	Unit Price	Total Cost
Land	lump sum	-	$ 40,000
Buildings	lump sum	-	40,000
Power, telephone & telegraph .	lump sum	-	4,000
Highways	1.5 mi.	-	40,000
Sub-Total - Reservoir Costs			$ 124,000
Engineering, Appraisals, Overhead & Contingencies (25%)			31,000
TOTAL RESERVOIR COSTS			$ 155,000

II. CONSTRUCTION COSTS

(a) Dam, Spillway & Outlets

	Quantity	Unit Price	Total Cost
Stream diversion & pumping	lump sum	-	$ 15,000
Clearing and grubbing. . . .	lump sum	-	6,500
Stripping	52,000 c.y.	$ 0.45	23,400
Excavation - common	306,000 c.y.	0.40	122,400
" - rock	48,000 c.y.	2.00	96,000
Borrow - pervious	45,000 c.y.	0.45	20,250
Sand for filters	800 c.y.	2.00	1,600
Gravel for filters & backing	6,700 c.y.	2.00	13,400
Structural backfill	13,000 c.y.	0.30	3,900
Rolled fill	210,000 c.y.	0.12	25,200
Rock fill (dumped)	55,000 c.y.	0.50	27,500
Derrick stone	4,500 c.y.	3.25	14,625
Drilling holes for grouting	3,000 l.f.	1.25	3,750
Pressure grouting	3,000 c.f.	1.25	3,750

II. CONSTRUCTION COSTS (continued)
(a) Dam, Spillway & Outlets (continued)

	Quantity	Unit Price	Total Cost
Line drilling	2,000 s.f.	$ 0.90	$ 1,800
Concrete - conduits	1,910 c.y.	16.50	31,515
" - walls and weir . .	13,500 c.y.	12.00	162,000
" - gate structures .	1,600 c.y.	16.50	26,400
" - spillway slab . .	4,800 c.y.	10.00	48,000
Reinforcing steel	600,000 lb.	0.05	30,000
Trash bars (installed) . . .	lump sum	-	2,000
Gates, guides, hoists (installed) . .	lump sum	-	6,500
Crane	lump sum	-	1,500
Operating superstructure . .	lump sum	-	5,000
Bridge, including pier . . .	lump sum	-	5,000
Drain wells	lump sum	-	10,000
Seeded topsoil	lump sum	-	6,500
Access road	lump sum	-	10,000
Miscellaneous	lump sum	-	14,510
TOTAL - DAM, SPILLWAY & OUTLETS			$ 738,000
(b) Reservoir Clearing	lump sum	-	14,000
Sub-Total - Construction Costs			$ 752,000
Engineering, Inspection, Overhead & Contingencies (35%±)			263,000
TOTAL - CONSTRUCTION COSTS			$1,015,000
TOTAL ESTIMATED COST			$1,170,000

2. The annual carrying charges on the above costs computed with interest rates at 3-1/2% amortization of fixed parts in 50 years and movable parts in 25 years, and allowing $4,000 annually for operation and maintenance are $54,100.

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APPENDIX G

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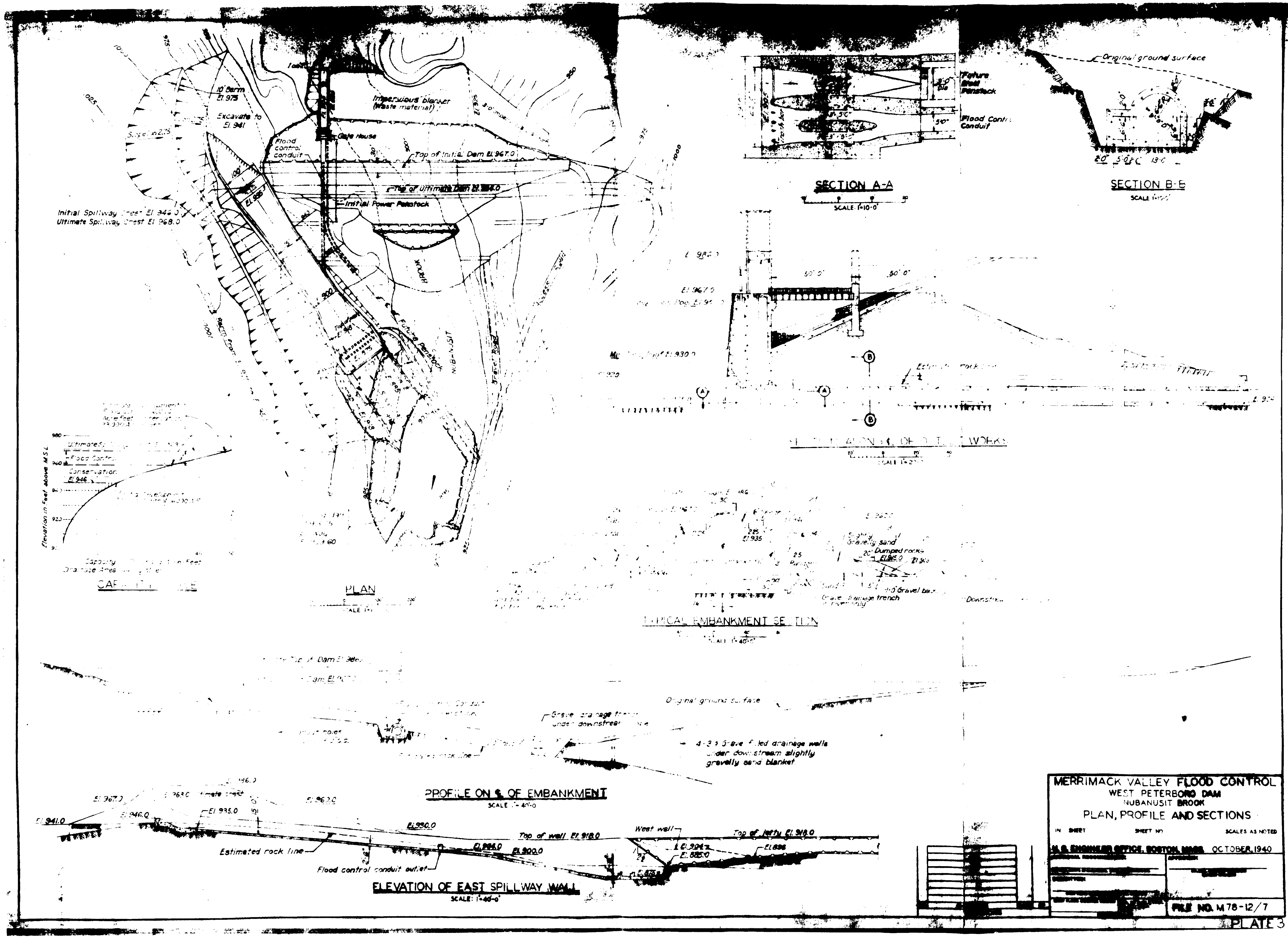
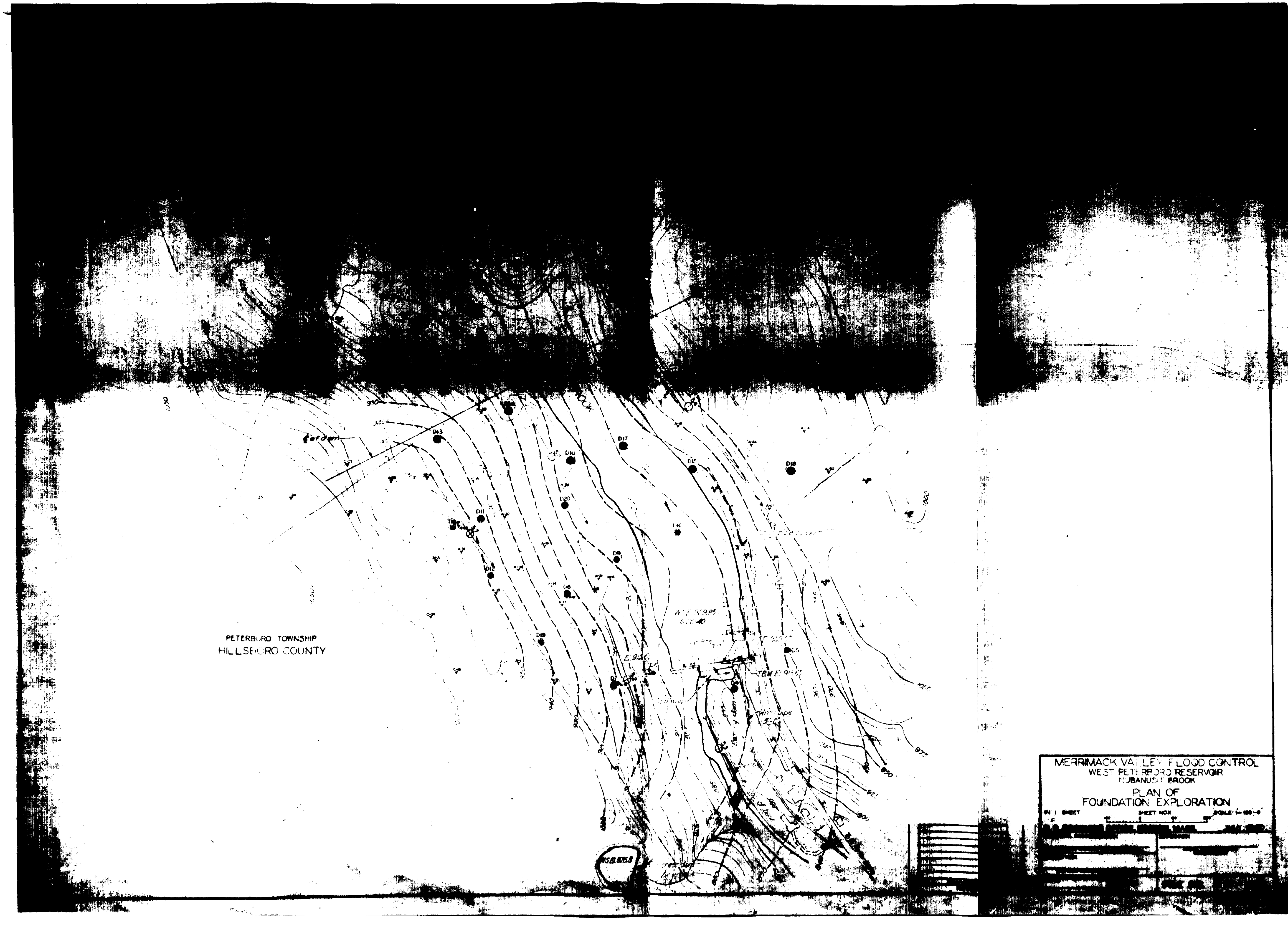
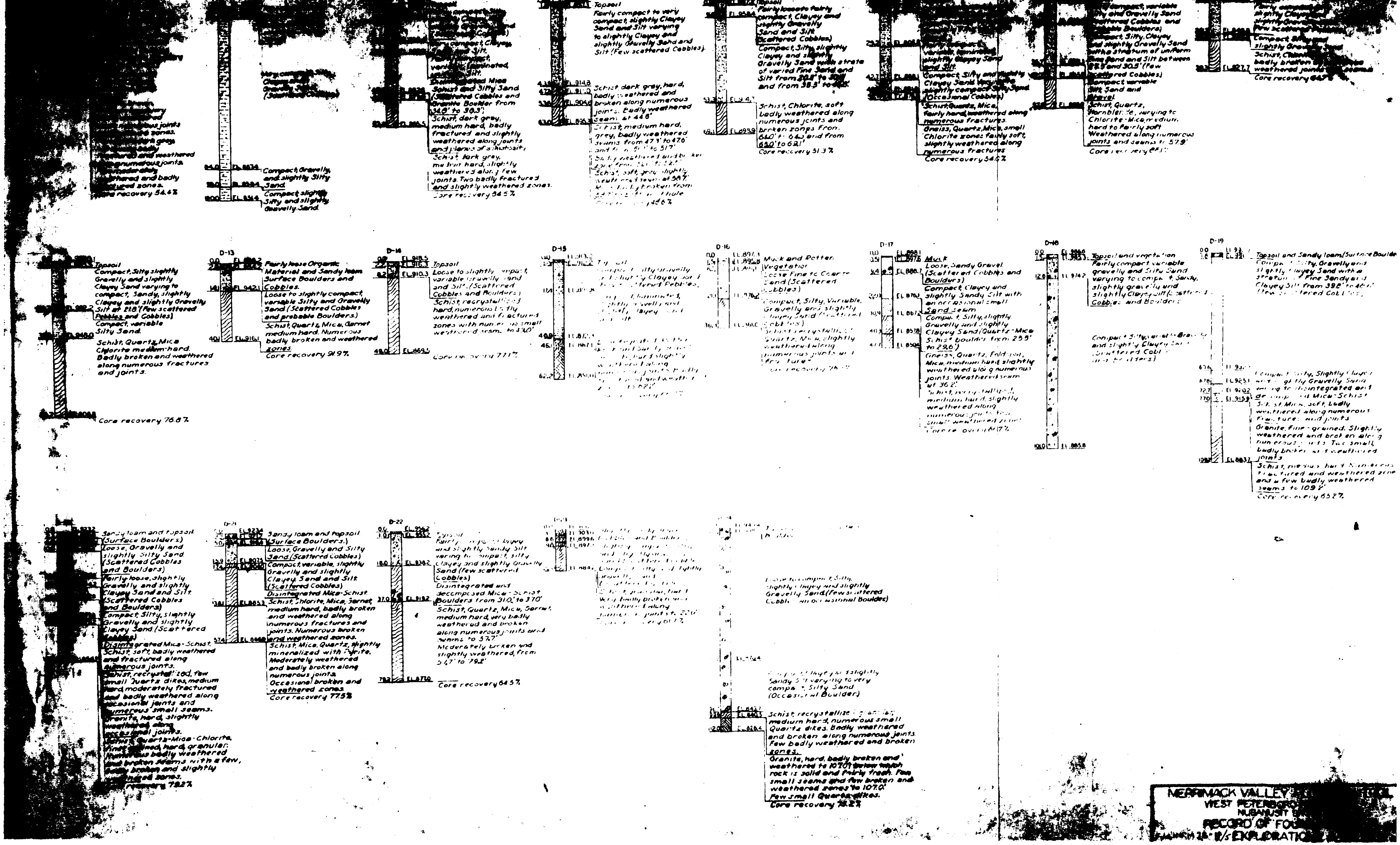


SECTION A-A
SCALE 1"=10'-0"
SECTION B-B
Original ground surface
Future Steel Penstock
Flood Control Conduit
Impervious blanket (Waste material)
Gate House
Flood control conduit
Top of Initial Dam El. 967.0
Initial Power Penstock
Future Penstock
Excavate to El. 941
10' Berm El. 975
Initial Spillway Crest El. 946.0
Ultimate Spillway Crest El. 968.0
HROOK
NUBANUSIT
PLAN
CAPACITY CURVE
Elevation in Feet above M.S.L.
Conservation El. 946
Slightly gravelly sand
20' Dumped rocks
Gravel drainage trench
TYPICAL EMBANKMENT SECTION
Original ground surface
4'-3" Gravel filled drainage wells under downstream slightly gravelly sand blanket
PROFILE ON ℄ OF EMBANKMENT
Estimated rock line
Flood control conduit outlet
Top of wall El. 918.0
West wall
Top of Jetty El. 918.0
ELEVATION OF EAST SPILLWAY WALL
SCALE: 1"=40'-0"
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO DAM
NUBANUSIT BROOK
PLAN, PROFILE AND SECTIONS
SCALES AS NOTED
U.S. ENGINEER OFFICE, BOSTON, MASS. OCTOBER, 1940
FILE NO. M 78-12/7
PLATE 3


PETERBORO TOWNSHIP
HILLSBORO COUNTY
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
PLAN OF
FOUNDATION EXPLORATION
D13
D17
D16
D15
D18
D11
D12
D8
D9
D19
D7
D6
D5
975
1000


D-13
Fairly loose Organic Material and Sandy loam Surface Boulders and Cobbles.
Loose to slightly compact, variable Silty and Gravelly Sand (Scattered Cobbles and probable Boulders.)
Schist, Quartz, Mica, Garnet medium hard. Numerous badly broken and weathered zones.
Core recovery 91.9%
D-14
Topsoil
Loose to slightly compact, variable Gravelly Sand and Silt (Scattered Cobbles and Boulders)
Core recovery 77.1%
D-15
D-16
Muck and Rotten Vegetation
Loose Fine to Coarse Sand (Scattered Cobbles)
Compact, Silty, Variable, Gravelly and slightly Clayey Sand
D-17
Muck
Loose, Sandy Gravel (Scattered Cobbles and Boulders)
Compact, Clayey and slightly Sandy Silt with an occasional small Sand seam
Compact, Silty, slightly Gravelly and slightly Clayey Sand (Quartz-Mica Schist boulder from 25.5' to 29.6')
D-18
Topsoil and vegetation
Fairly compact, variable gravelly and Silty Sand varying to compact, Sandy, slightly gravelly and slightly Clayey Silt (Scattered Cobbles and Boulders)
D-19
Topsoil and Sandy loam (Surface Boulders)
Granite, Fine-grained. Slightly weathered and broken along numerous joints.
Core recovery 65.8%
D-21
Sandy loam and topsoil (Surface Boulders.)
Loose, Gravelly and Silty Sand (Scattered Cobbles)
Compact, variable, slightly Gravelly and slightly Clayey Sand and Silt (Scattered Cobbles)
Disintegrated Mica-Schist
Schist, Chlorite, Mica, Garnet, medium hard, badly broken and weathered along numerous fractures and joints. Numerous broken and weathered zones.
Schist, Mica, Quartz, slightly mineralized with Pyrite. Moderately weathered and badly broken along numerous joints.
Occasional broken and weathered zones.
Core recovery 77.5%
D-22
Topsoil
Disintegrated and decomposed Mica-Schist Boulders from 31.0' to 37.0'
Schist, Quartz, Mica, Garnet, medium hard, very badly weathered and broken along numerous joints and seams to 54.7'
Moderately broken and slightly weathered, from 54.7' to 79.2'
Core recovery 64.5%
Loose to compact, Silty, slightly Clayey and slightly Gravelly Sand (Few scattered Cobbles - an occasional Boulder)
Schist, recrystallized, medium hard, numerous small Quartz dikes. Badly weathered and broken along numerous joints.
Few badly weathered and broken zones.
Granite, hard, badly broken and weathered to 107.0' below which rock is solid and fairly fresh. Few small seams and few broken and weathered zones to 107.0'.
Few small Quartz dikes.
Core recovery 78.8%
Core recovery 76.8%
Core recovery 54.5%
Core recovery 51.3%
Core recovery 54.6%
MERRIMACK VALLEY
WEST PETERBOROUGH
NUBANUSIT
RECORD OF FOUNDATION EXPLORATIONS

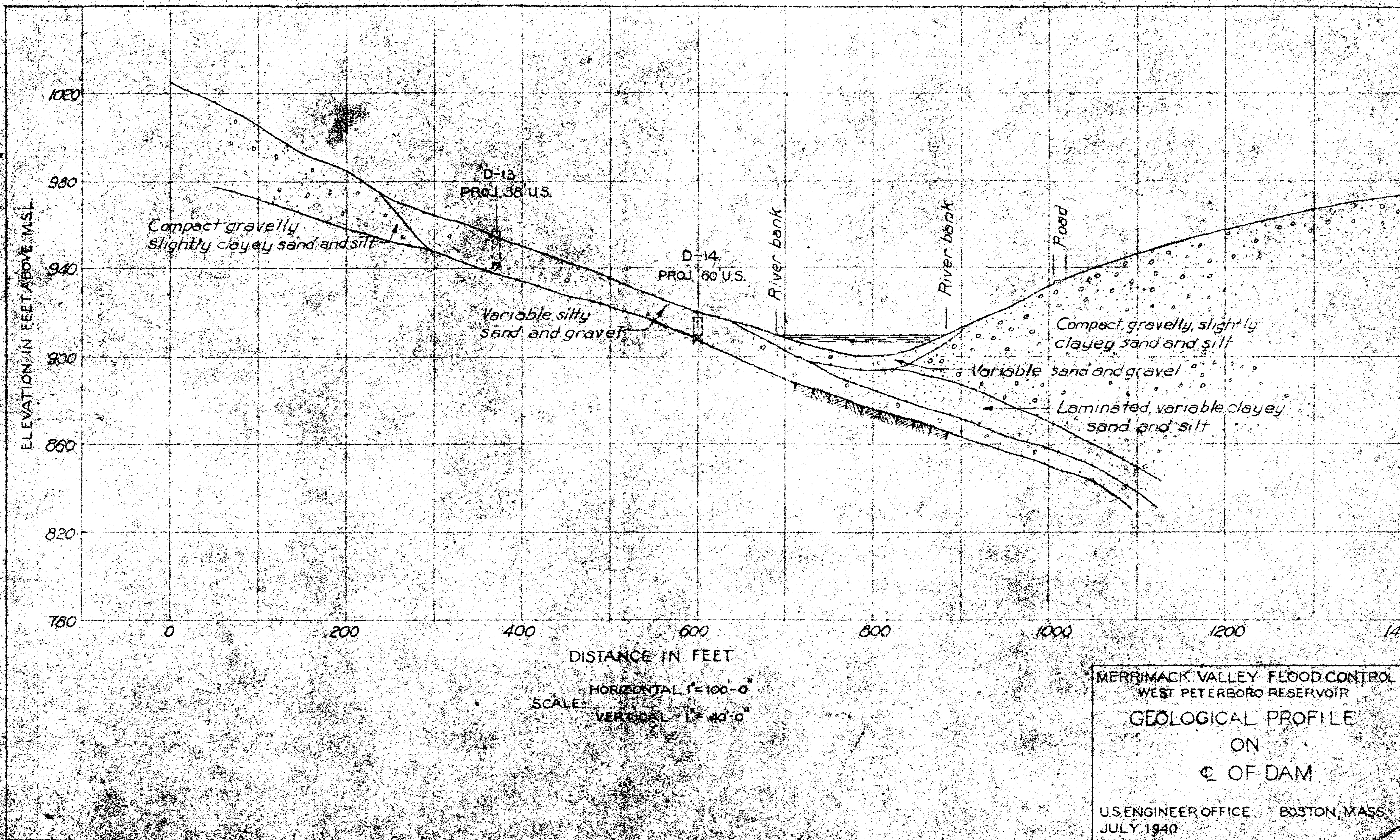

ELEVATION IN FEET ABOVE MSL
1020
980
940
900
860
820
780
0
200
400
600
800
1000
1200
1400
DISTANCE IN FEET
SCALE: HORIZONTAL 1" = 100'-0" VERTICAL 1" = 40'-0"
D-13 PROJ. 38' U.S.
D-14 PROJ. 60' U.S.
Compact gravelly slightly clayey sand and silt
Variable silty sand and gravel
River bank
River bank
Road
Compact, gravelly, slightly clayey sand and silt
Variable sand and gravel
Laminated, variable clayey sand and silt
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
GEOLOGICAL PROFILE
ON
℄ OF DAM
U.S. ENGINEER OFFICE BOSTON, MASS.
JULY 1940

PLATE 6

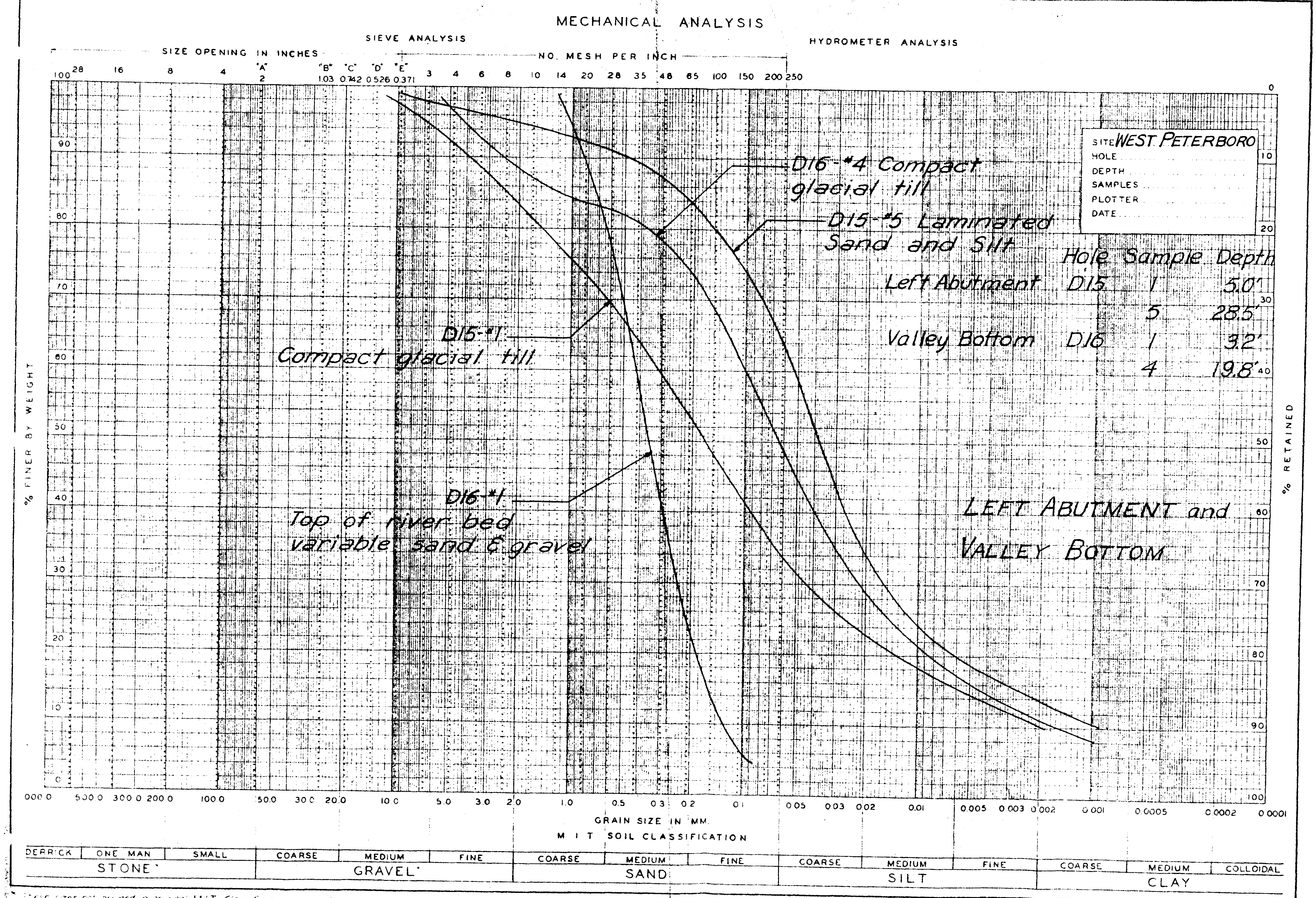
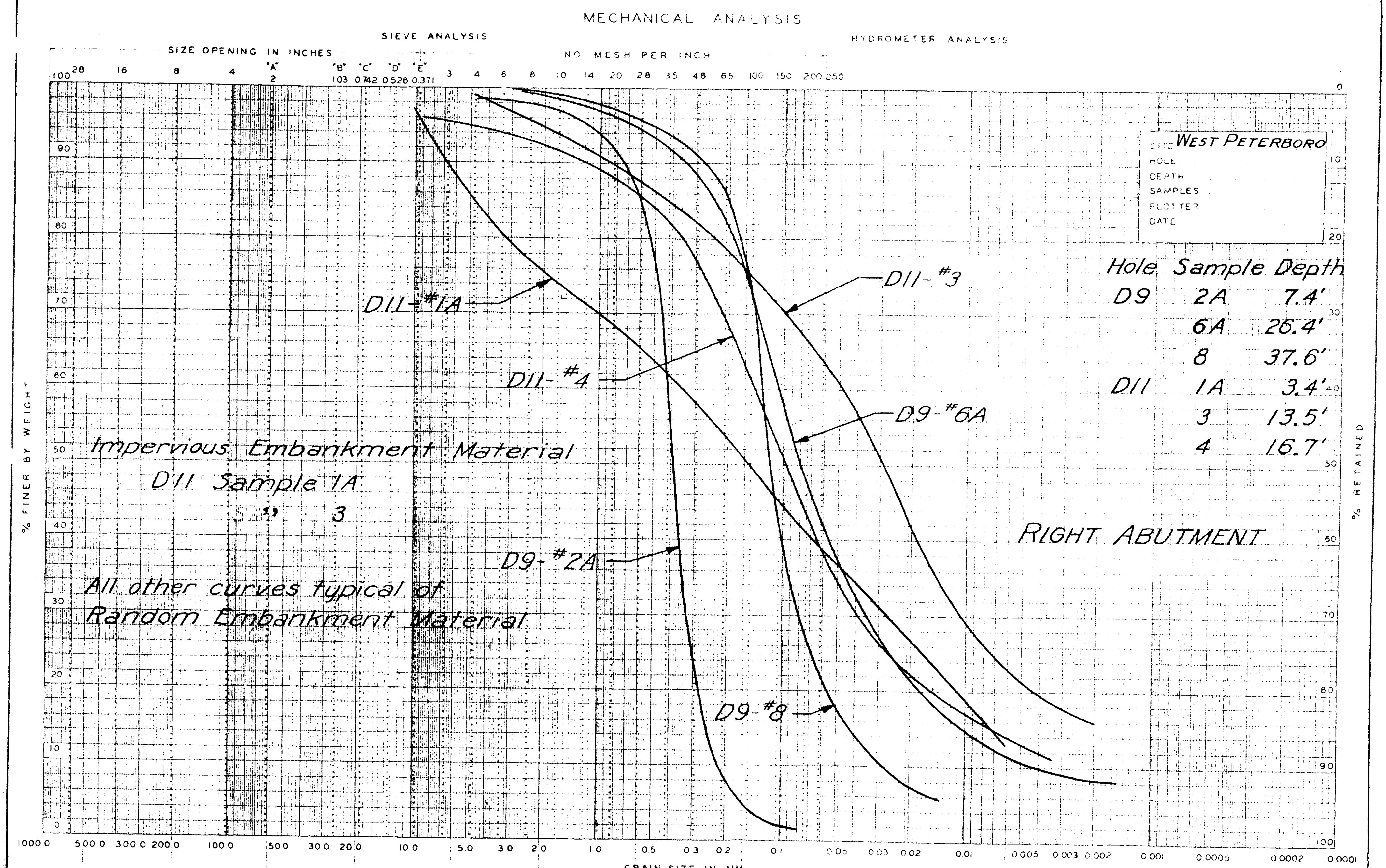


PLATE 7

SL-5.00



DERRICK	ONE MAN	SMALL	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	COLLOIDAL
STONE*			GRAVEL*			SAND			SILT			CLAY		

* These sizes not included in original M.I.T. Classification

PLATE 8

SL-5.00

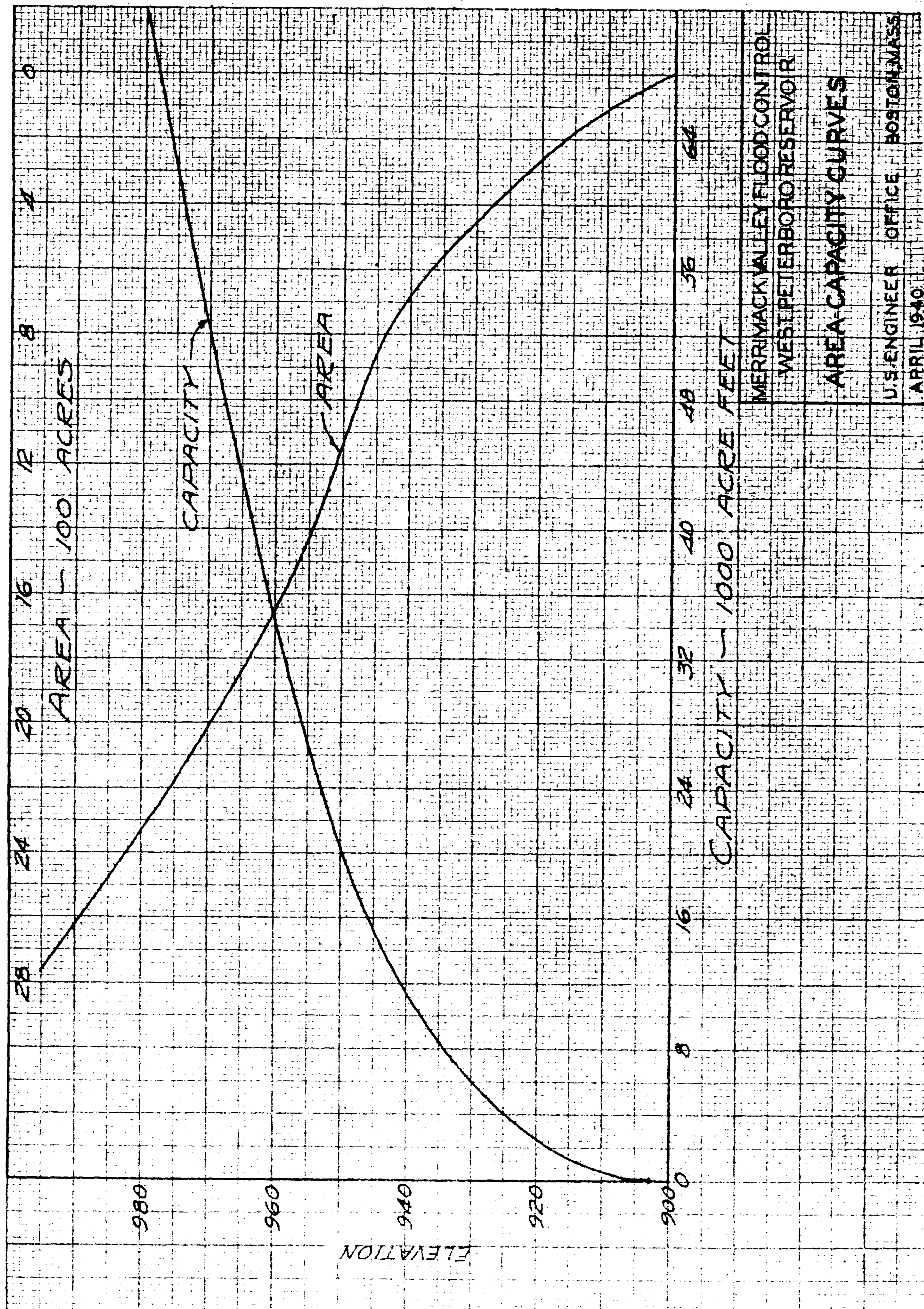
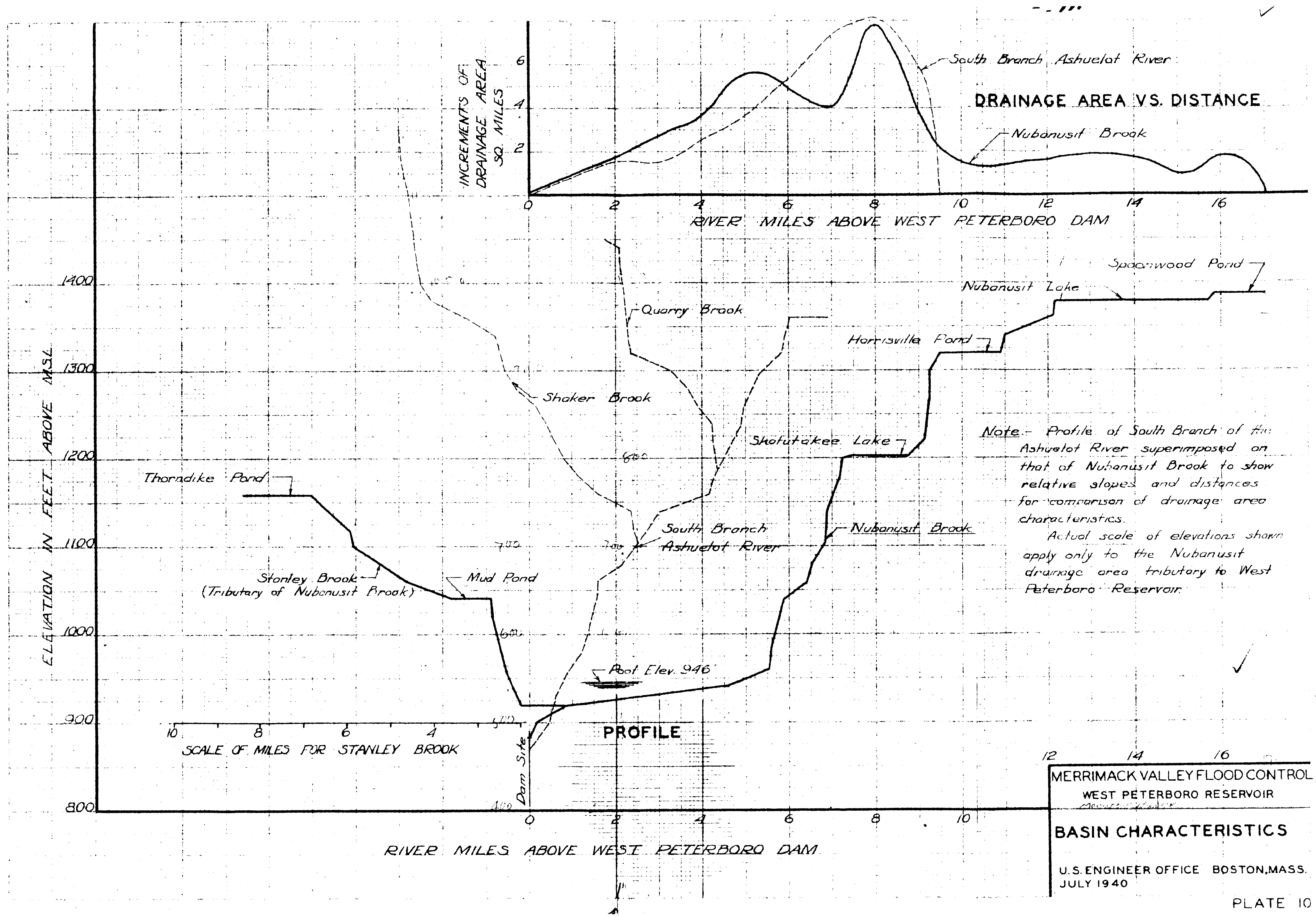


PLATE 9

ticut, is a member

Willi... Wise, 37 Bishop Road, West Hartford, Connec



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

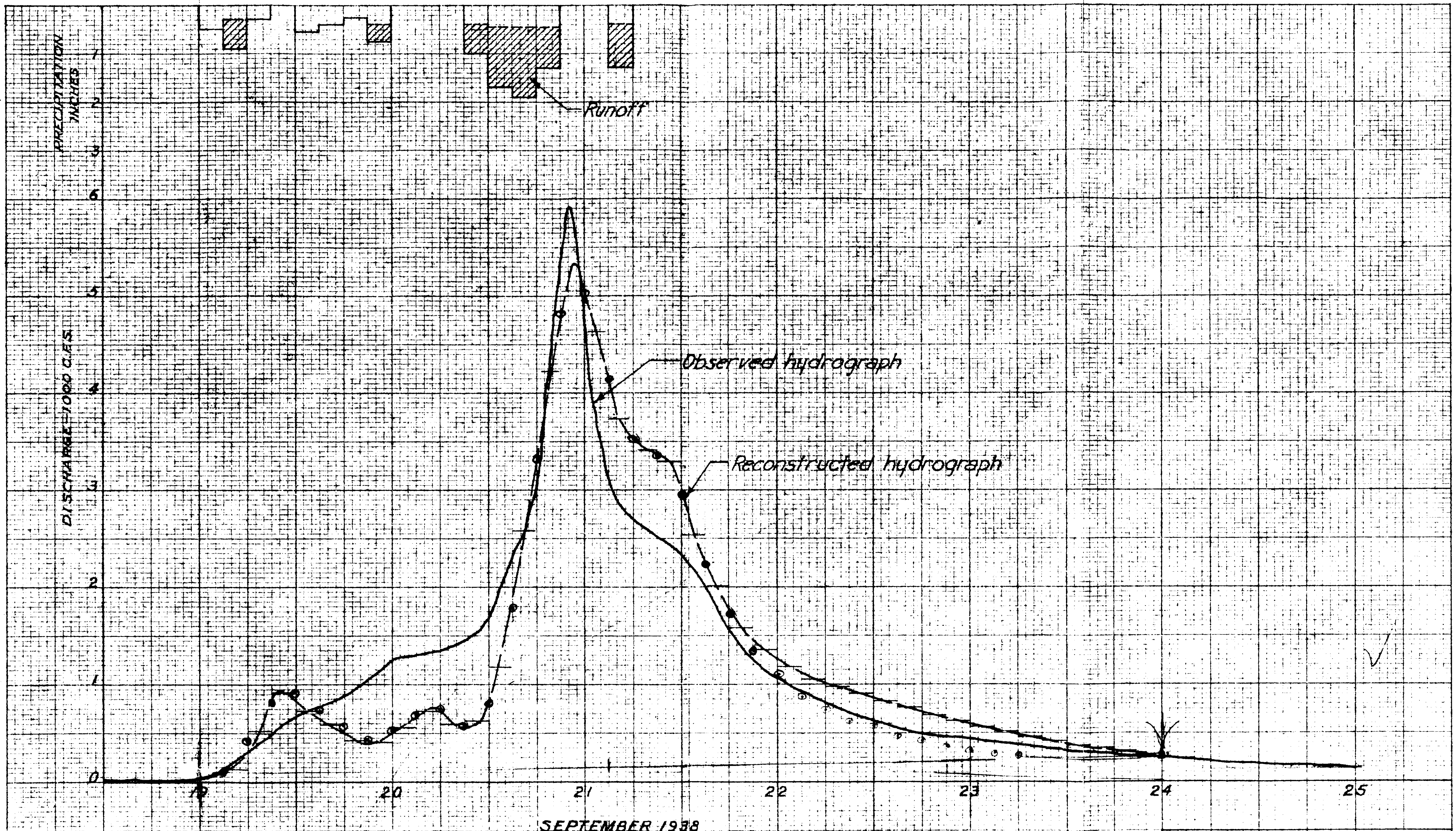
BASIN CHARACTERISTICS

U.S. ENGINEER OFFICE BOSTON, MASS.
JULY 1940

PLATE 10

WAR DEPARTMENT CORPS OF ENGINEERS, U.S. ARMY

PLATE 11

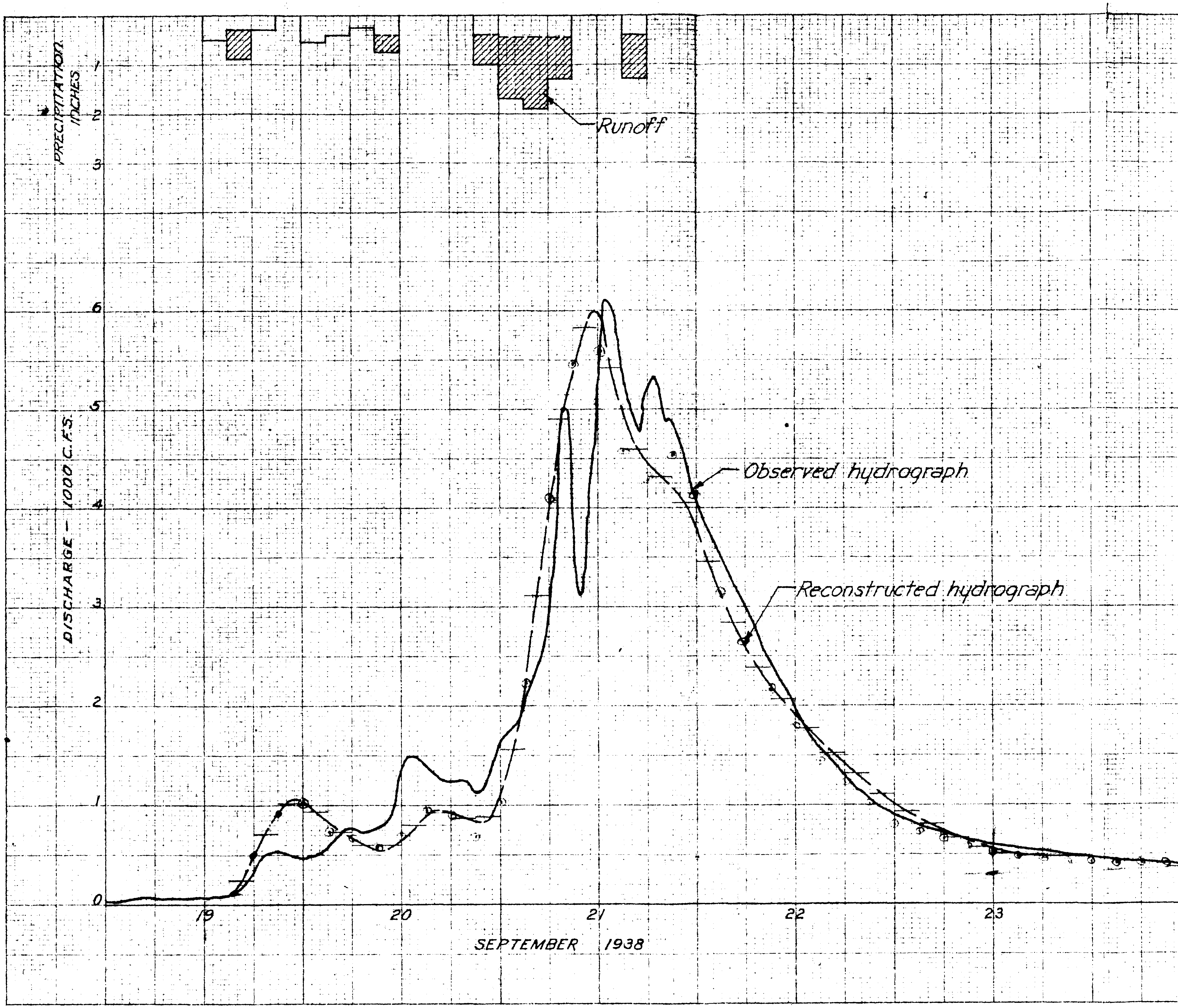


MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

ANALYSIS OF
THE 1938 FLOOD
AT THE SOUTH BRANCH
ASHUELOT RIVER

U.S. ENGINEER OFFICE, BOSTON, MASS.
JULY, 1940.

PLATE 12

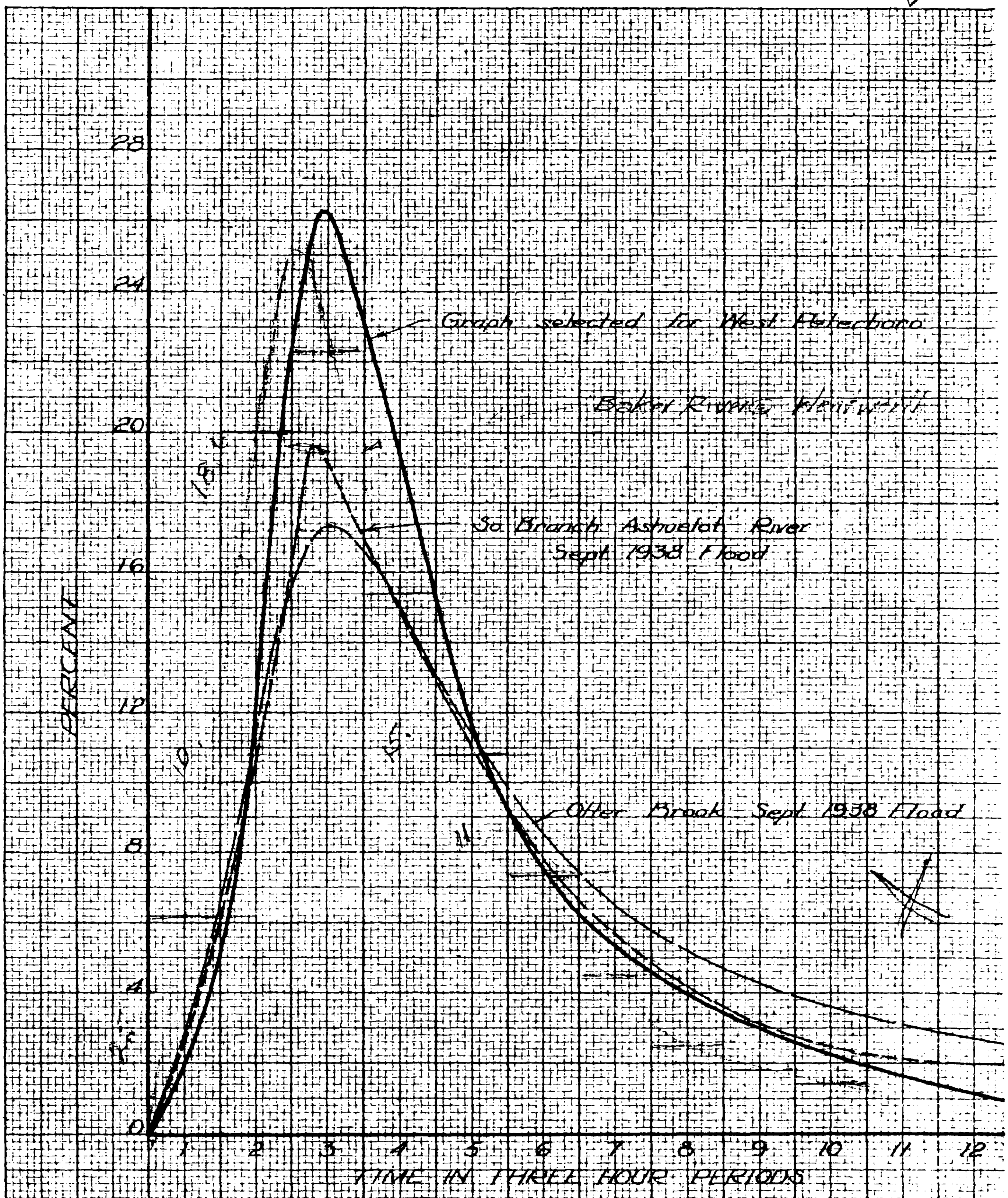


MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

ANALYSIS OF THE 1938 FLOOD AT OTTER BROOK

U. S. ENGINEER OFFICE, BOSTON, MASS.
JULY, 1940.

PLATE 13

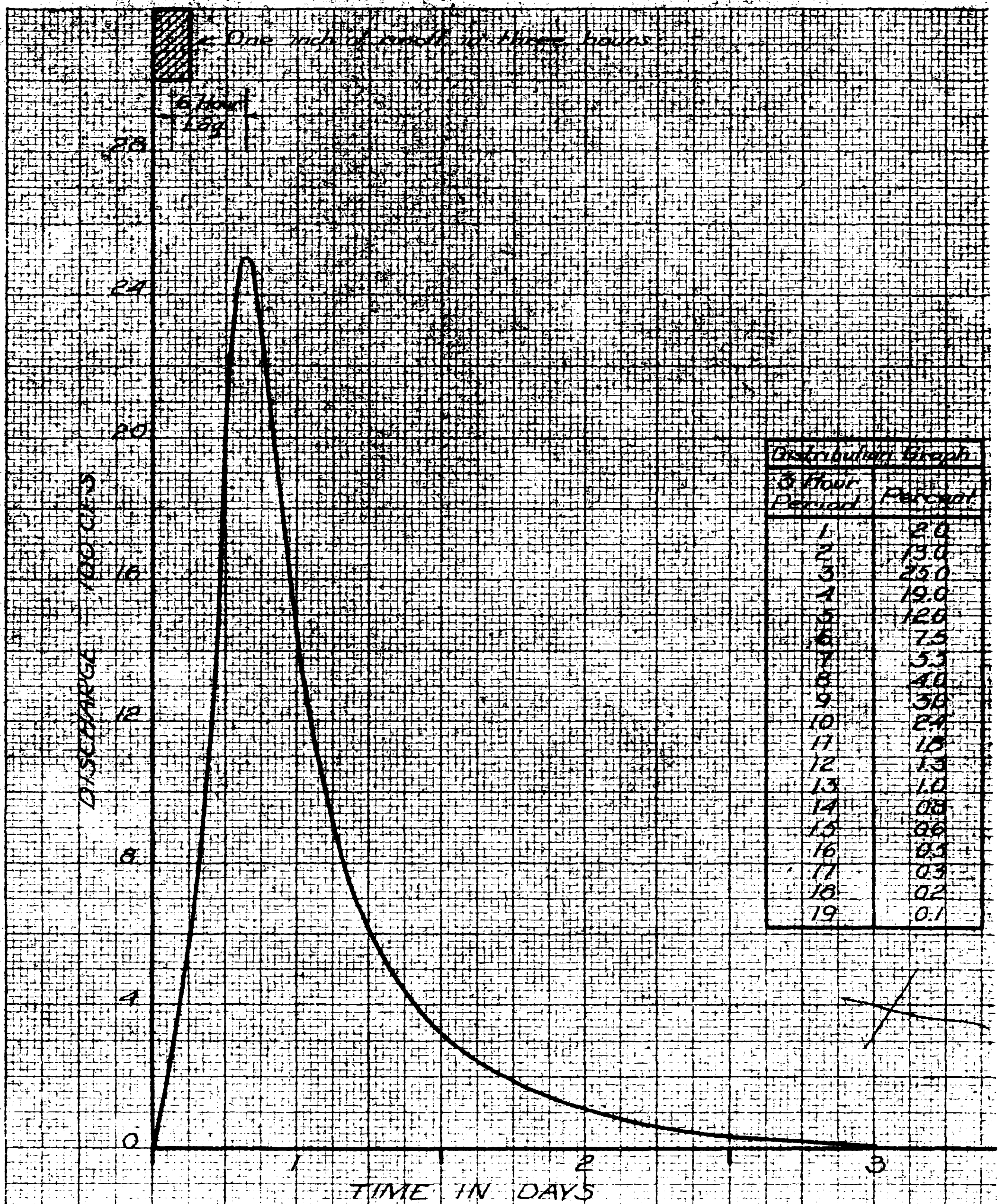


MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

DISTRIBUTION GRAPHS

U.S. ENGINEER OFFICE BOSTON, MASS.
JULY 1940

PLATE 14



Distribution Graph	
3 Hour Period	Percent
1	2.0
2	13.0
3	25.0
4	19.0
5	12.0
6	7.5
7	5.3
8	4.0
9	3.0
10	2.4
11	1.8
12	1.3
13	1.0
14	0.8
15	0.6
16	0.5
17	0.3
18	0.2
19	0.1

MERRIMACK VALLEY FLOOD CONTRO

WEST PETERBORO RESERVOIR

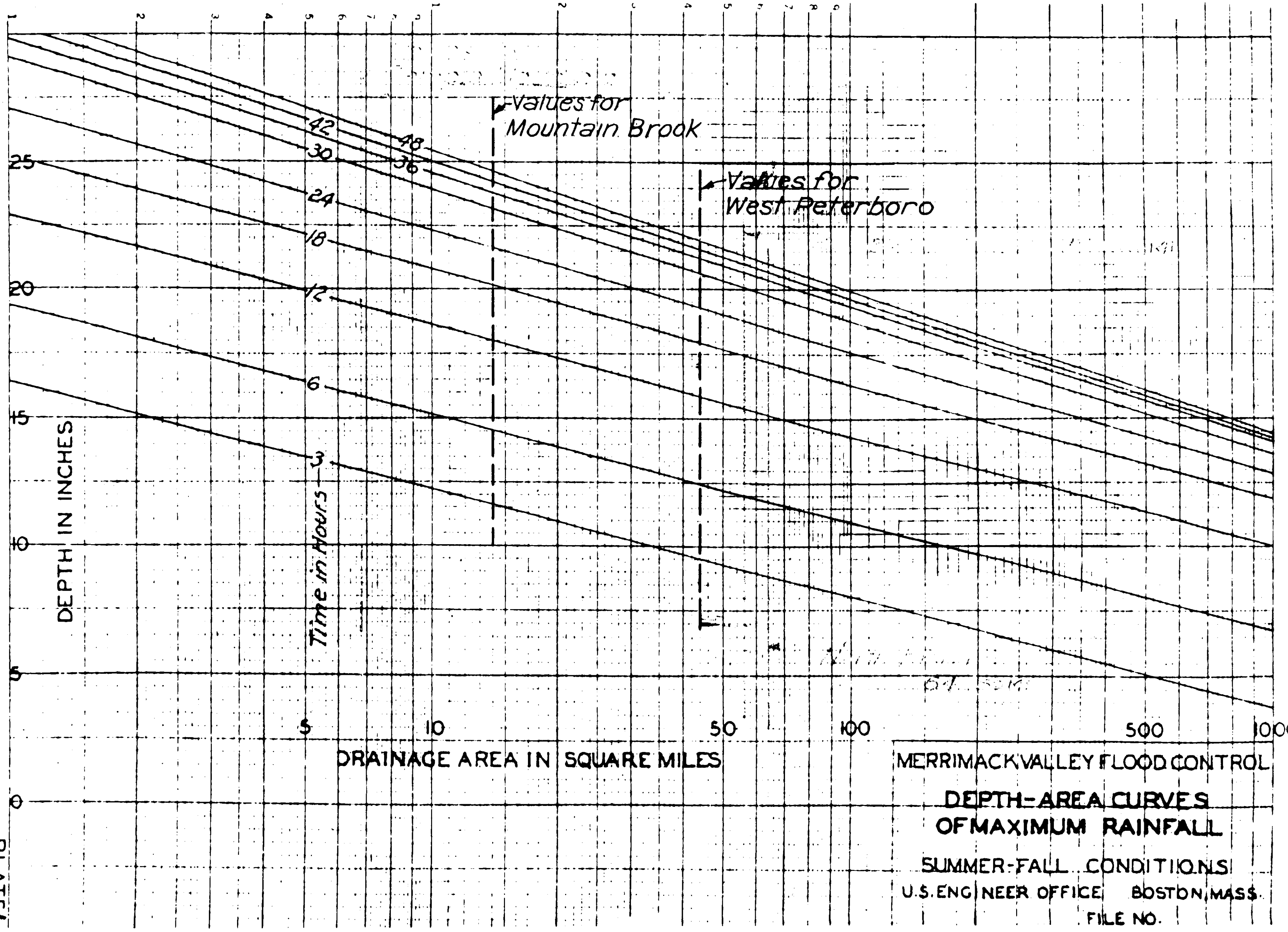
INFLOW UNIT HYDROGRAPH

U. S. ENGINEER OFFICE, BOSTON, MASS

JULY 1940

PLATE I

MADE IN U.S.A.

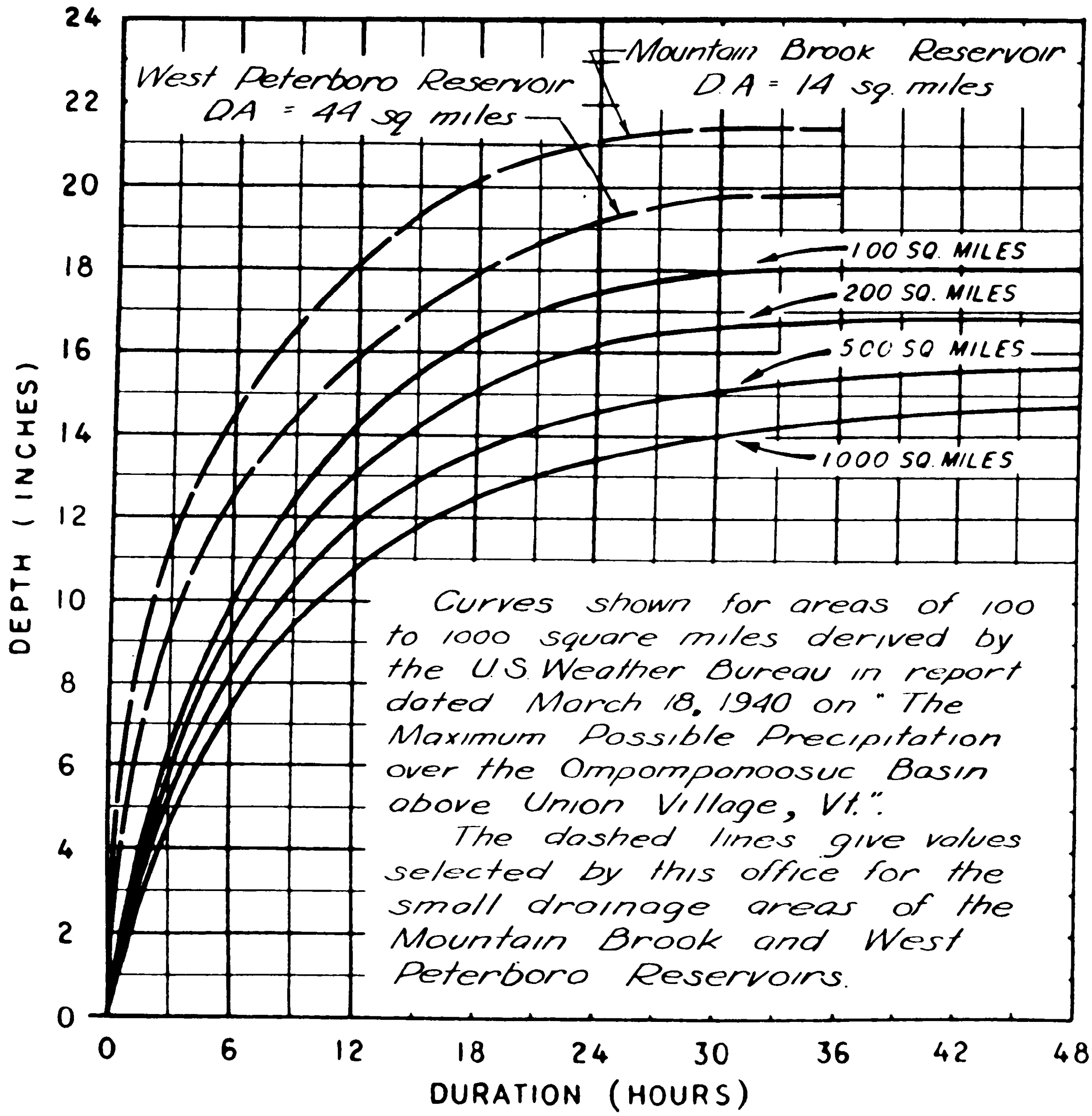

Values for Mountain Brook
Values for West Peterboro
42
48
30
36
24
18
12
6
3
Time in Hours
DEPTH IN INCHES
25
20
15
10
5
0
5
10
50
100
500
1000
DRAINAGE AREA IN SQUARE MILES
MERRIMACK VALLEY FLOOD CONTROL
DEPTH-AREA CURVES
OF MAXIMUM RAINFALL
SUMMER-FALL CONDITIONS
U.S. ENGINEER OFFICE BOSTON, MASS.
FILE NO.

PLATE

U. S. DEPT. OF AGRICULTURE | RIVER & FLOOD DIVISION
WEATHER BUREAU | HYDROMETEOROLOGICAL SECTION

ENVELOPING DURATION-DEPTH CURVES OF MAXIMUM POSSIBLE RAINFALL

OVER SELECTED BASINS IN THE NEW ENGLAND REGION



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

RAINFALL DATA FOR SPILLWAY DESIGN

U. S. ENGINEER OFFICE BOSTON, MASS.
JULY 1940

PLATE 1?

WAR DEPARTMENT

U. S. ENGINEER OFFICE, BOSTON, MASS.

Page

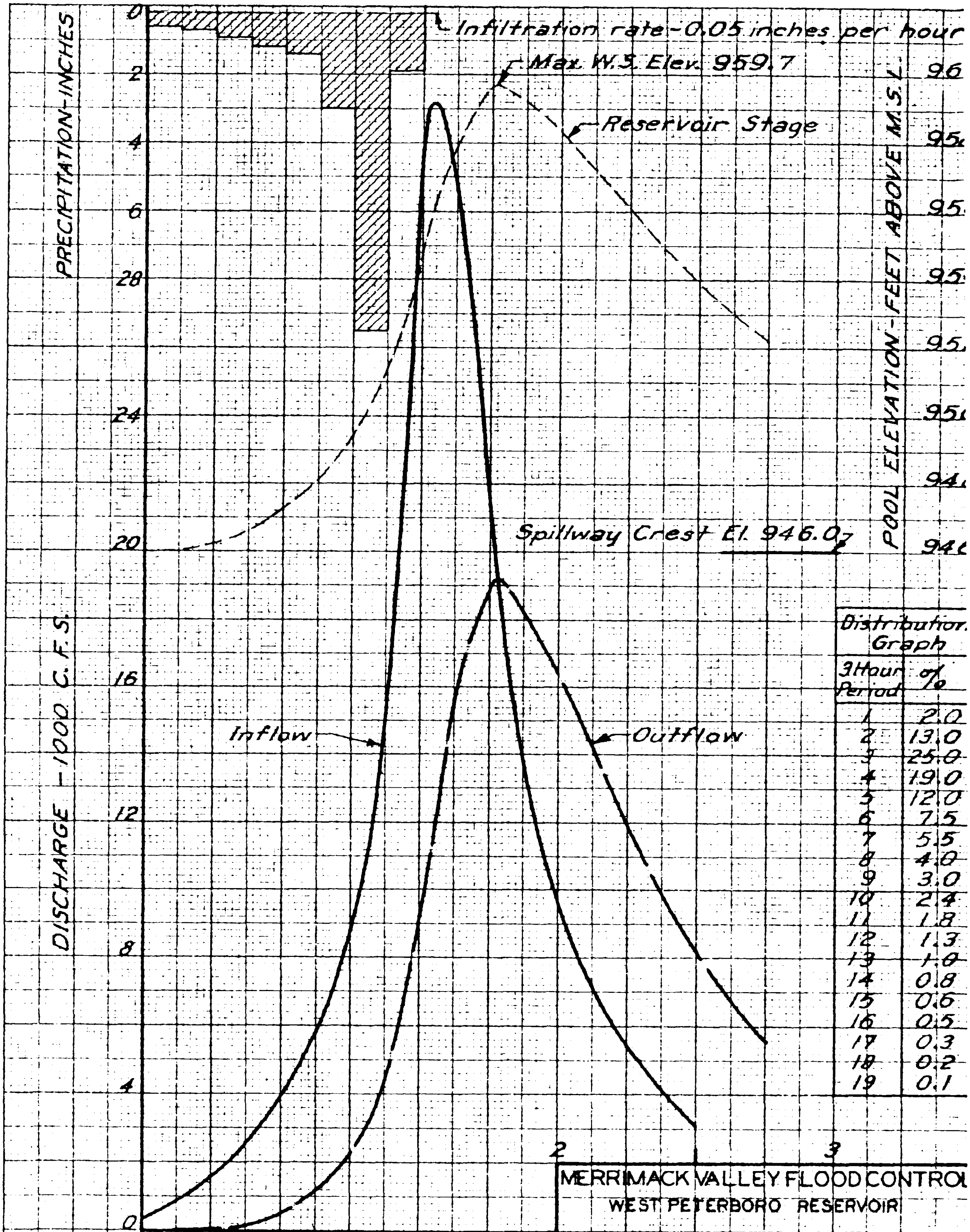
West Peterboro Reservoir

Computed Spillway Flood - Summer-Fall Conditions

Checked by   Date

Time	Rainfall Inches	0.05"/hr Infiltration Inches	Runoff Inches	Runoff csf/18 day	Dist. Values %	Distributed Runoff 1	2	3	4	5	6	7	8	Total Runoff + 220 cfs Base Flow
M														
3	0.60	0.15	0.45	4259	2.0	85								305
6	0.70	0.15	0.55	5205	13.0	554	104							878
9	0.90	0.15	0.75	7098	25.0	1065	677	142						2104
1 N	1.20	0.15	1.05	9937	19.0	809	1301	923	199					3452
3	1.40	0.15	1.25	11830	12.0	511	989	1775	1292	237				5024
6	3.00	0.15	2.85	26972	7.5	319	625	1349	2484	1538	539			7074
9	9.50	0.15	9.35	88488	5.5	234	390	852	1888	2958	3506	1770		11715
M	1.90	0.15	1.75	16562	4.0	170	286	532	1192	2248	6743	11530	331	23225
3					3.0	128	208	390	745	1420	5125	22122	2153	32511
6					2.4	102	156	284	547	887	3237	16813	4141	26387
9					1.8	77	125	213	397	651	2023	10619	3147	17472
2 N					1.3	55	94	170	298	473	1483	6637	1987	11417
3					1.0	43	68	128	238	355	1079	4867	1242	8240
6					0.8	34	52	92	179	284	809	3540	911	6121
9					0.6	26	42	71	129	213	647	2655	662	4665
M					0.5	21	31	57	99	154	485	2124	497	3688
3					0.3	13	26	43	79	118	351	1593	397	2840
6					0.2	9	16	35	60	95	270	1150	298	2153
9					0.1	4	10	21	50	71	216	885	215	1692
3 N							5	14	30	59	162	708	166	1364
3					✓			7	20	35	135	531	132	1080
6									10	24	81	442	99	876
9										12	54	265	83	634
M											27	177	50	474
												88	33	341
													17	237
	19.20	1.20	18.00	170351	100.0	4259	5205	7098	9936	11832	26977	88488	16561	171119

PLATE 18



Distribution Graph	
3 Hour Period	%
1	2.0
2	13.0
3	25.0
4	19.0
5	12.0
6	7.5
7	5.5
8	4.0
9	3.0
10	2.4
11	1.8
12	1.3
13	1.0
14	0.8
15	0.6
16	0.5
17	0.3
18	0.2
19	0.1

Note:
Based on maximum
Summer-Fall conditions

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

COMPUTED SPILLWAY FLOOD
INFLOW—OUTFLOW
STAGE HYDROGRAPH

U.S. ENGINEER OFFICE BOSTON, MASS.
JULY 1940

PLATE 19

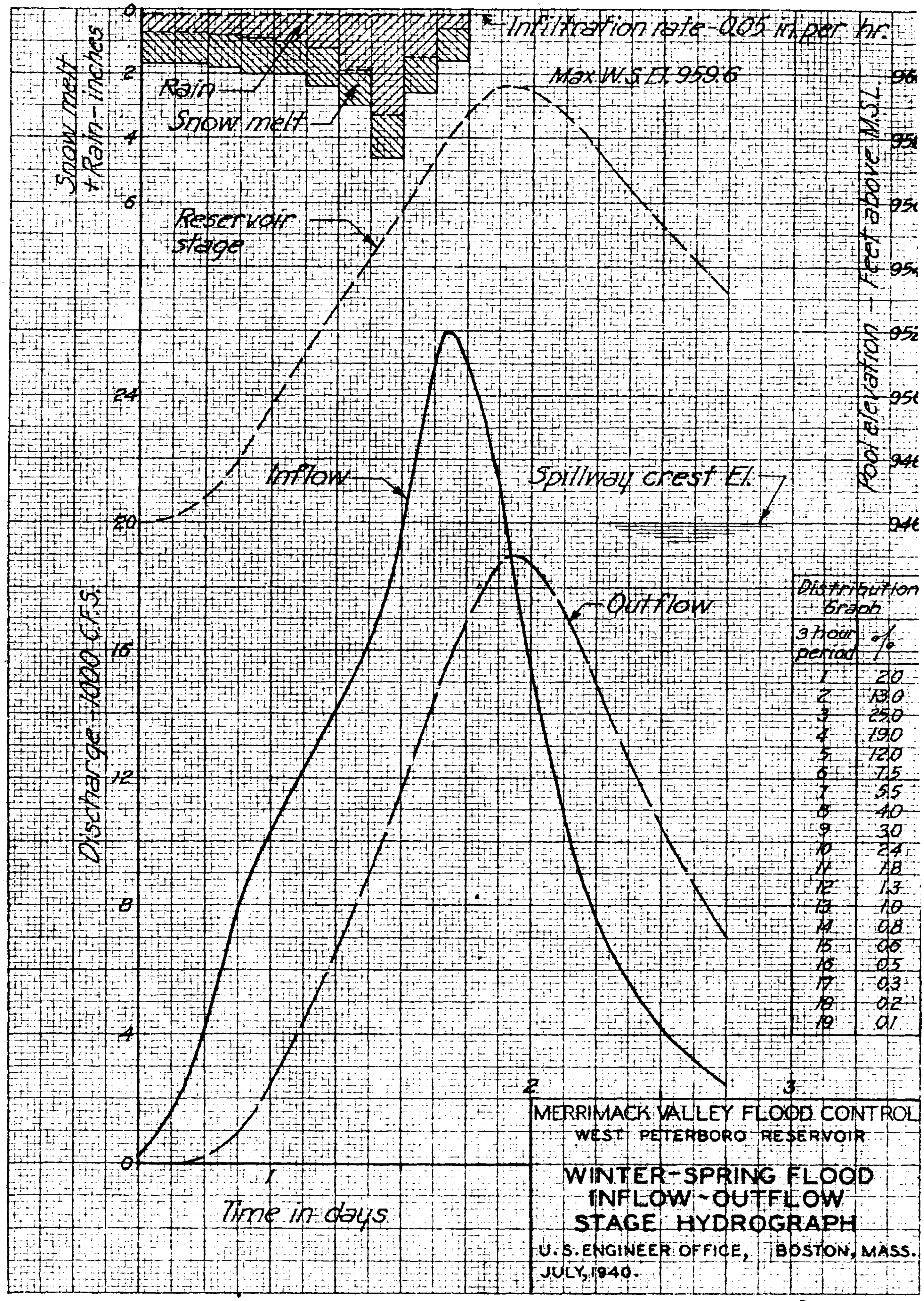
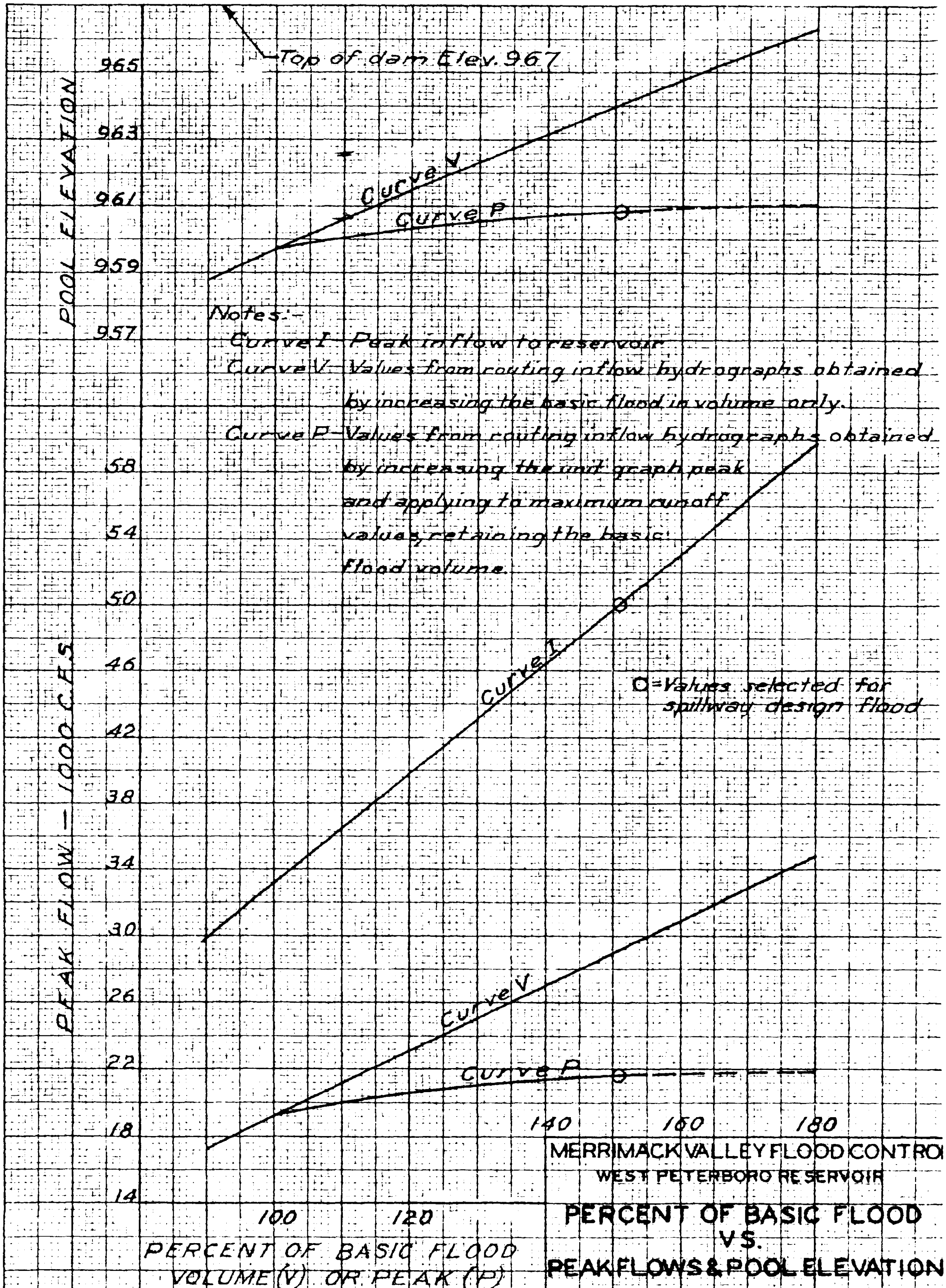

Infiltration rate - 0.05 in. per hr.
Max. W.S. El. 959.6
Rain
Snow melt
Snow melt + Rain - inches
Reservoir stage
Inflow
Outflow
Spillway crest El.
Pool elevation - Feet above M.S.L.
Discharge - 1000 C.F.S.
Time in days
Distribution Graph
3 hour period
%
1 2.0
2 13.0
3 25.0
4 19.0
5 12.0
6 7.5
7 5.5
8 4.0
9 3.0
10 2.4
11 1.8
12 1.3
13 1.0
14 0.8
15 0.6
16 0.5
17 0.3
18 0.2
19 0.1
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
WINTER-SPRING FLOOD
INFLOW - OUTFLOW
STAGE HYDROGRAPH
U. S. ENGINEER OFFICE, BOSTON, MASS.
JULY, 1940.

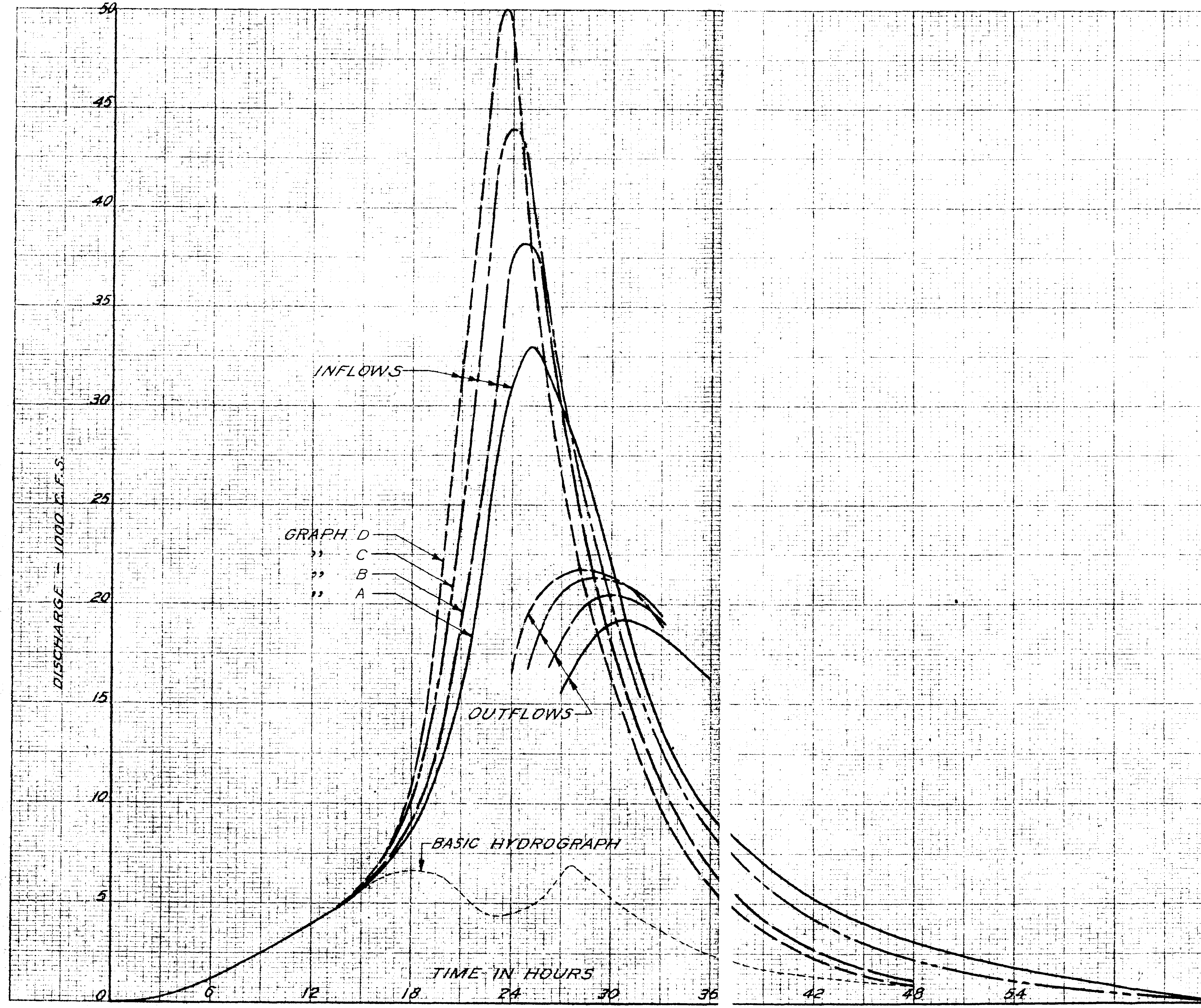
PLATE 20



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

PERCENT OF BASIC FLOOD
VS.
PEAK FLOWS & POOL ELEVATION

U.S. ENGINEER OFFICE — BOSTON, MASS
JULY 1940



MERRIMACK VALLEY FLOOD CONTR

WEST PETERBORO RESERVOIR

COMPARISON OF

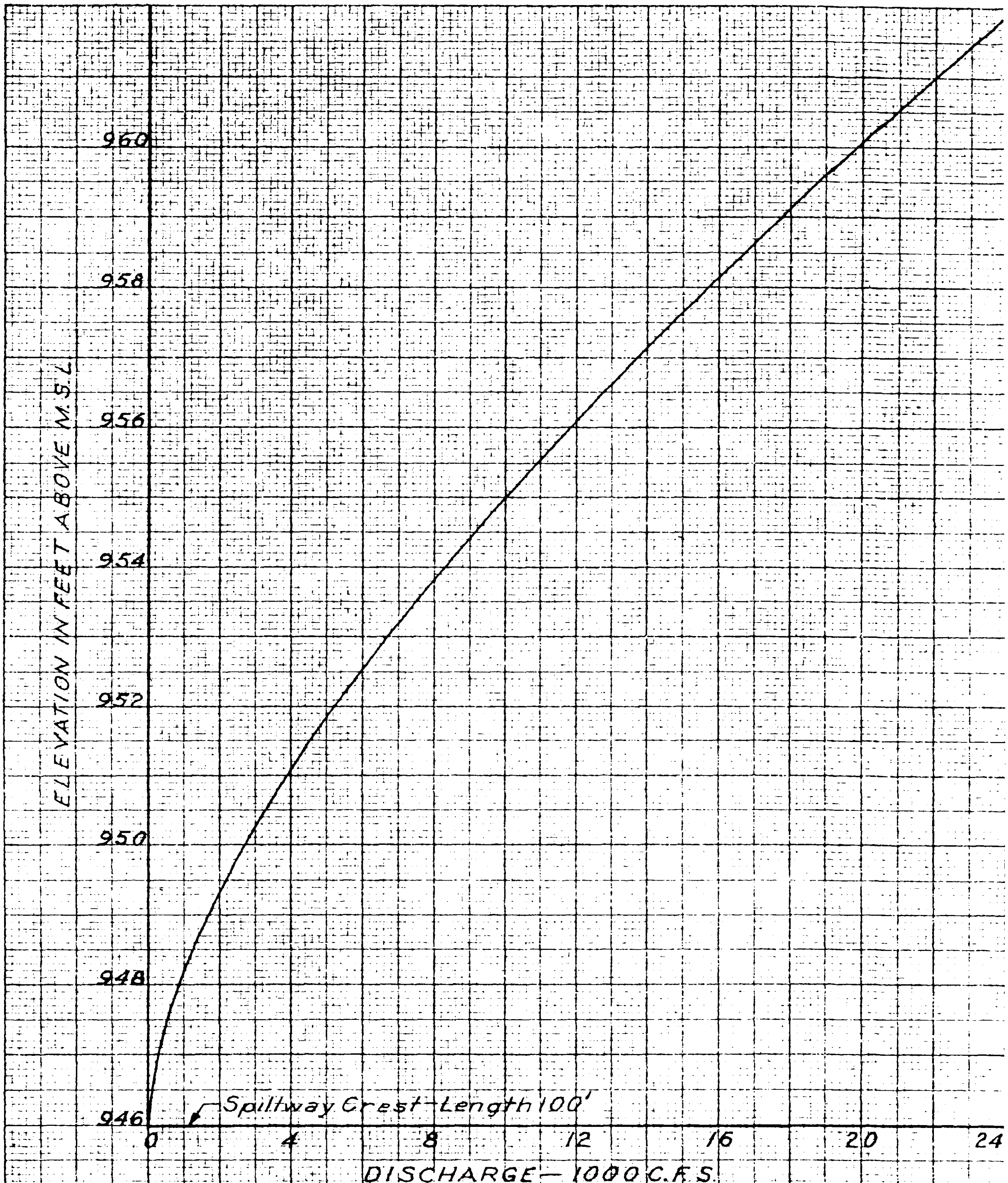
HYPOTHETICAL HYDROGRAPH

FOR SPILLWAY DESIGN FLOOD

U.S. ENGINEER OFFICE BOSTON, MAS

JULY 1940

PLATE



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

SPILLWAY RATING CURVE

U.S. ENGINEER OFFICE BOSTON, MASS.
JULY 1940

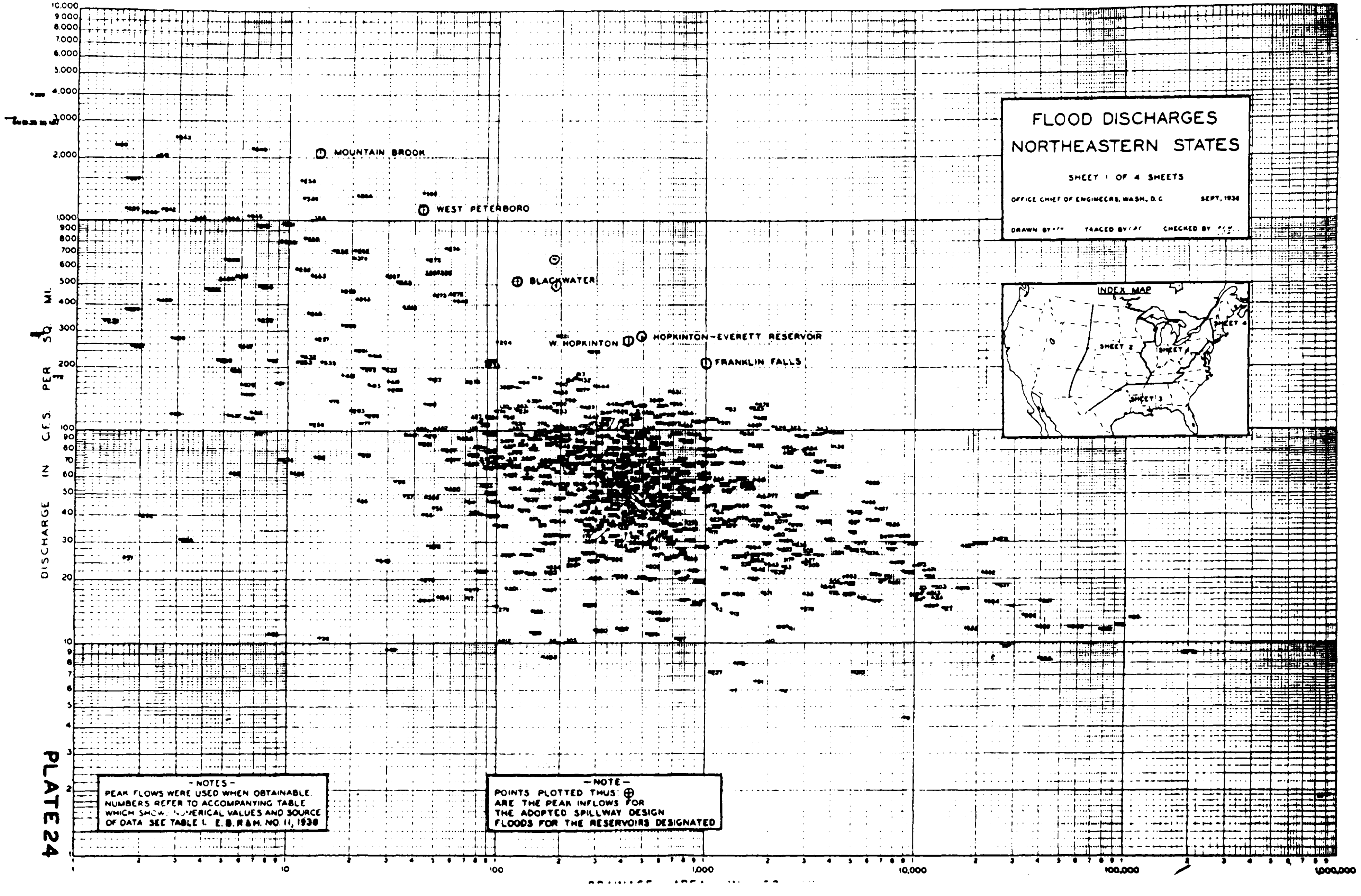
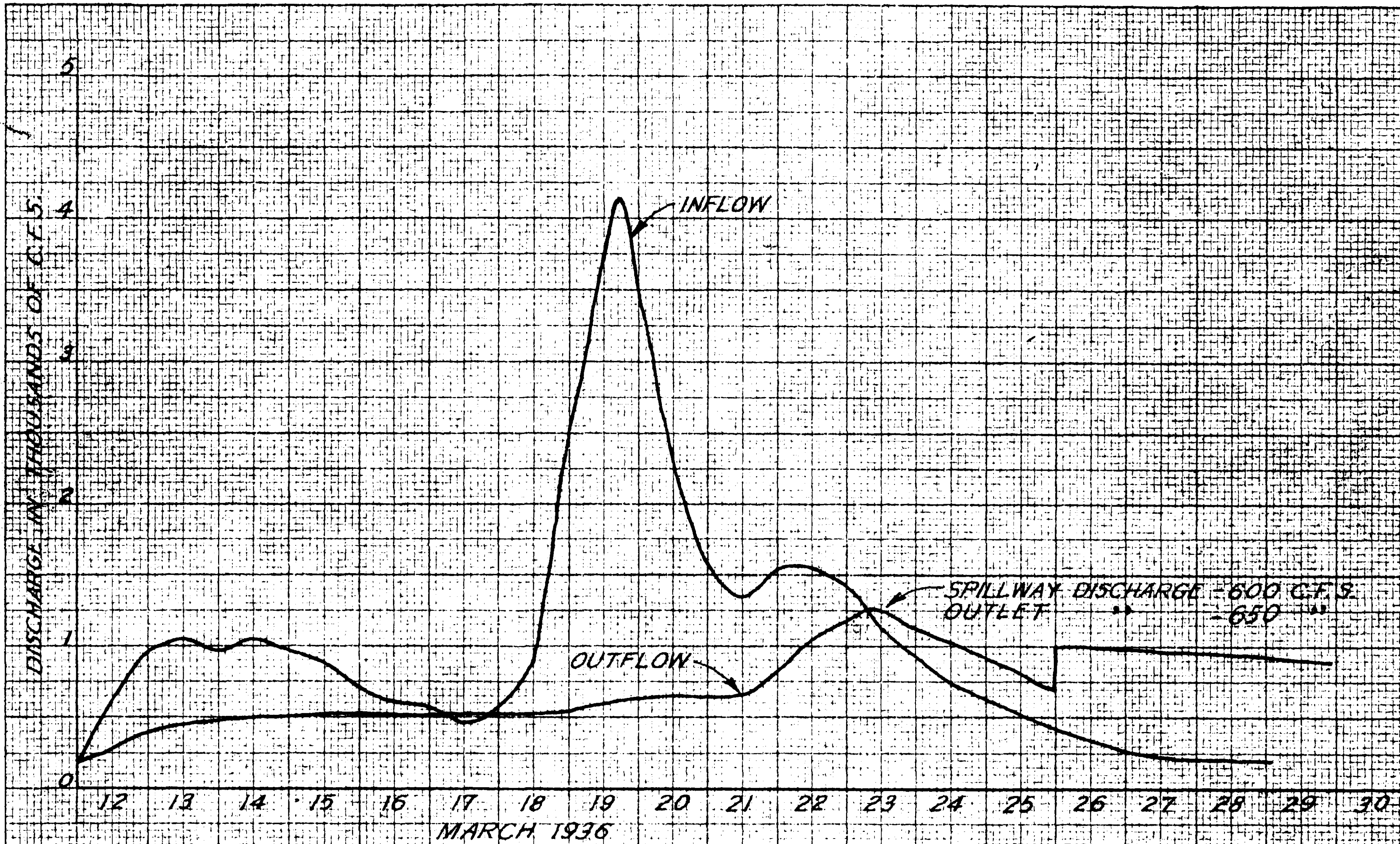
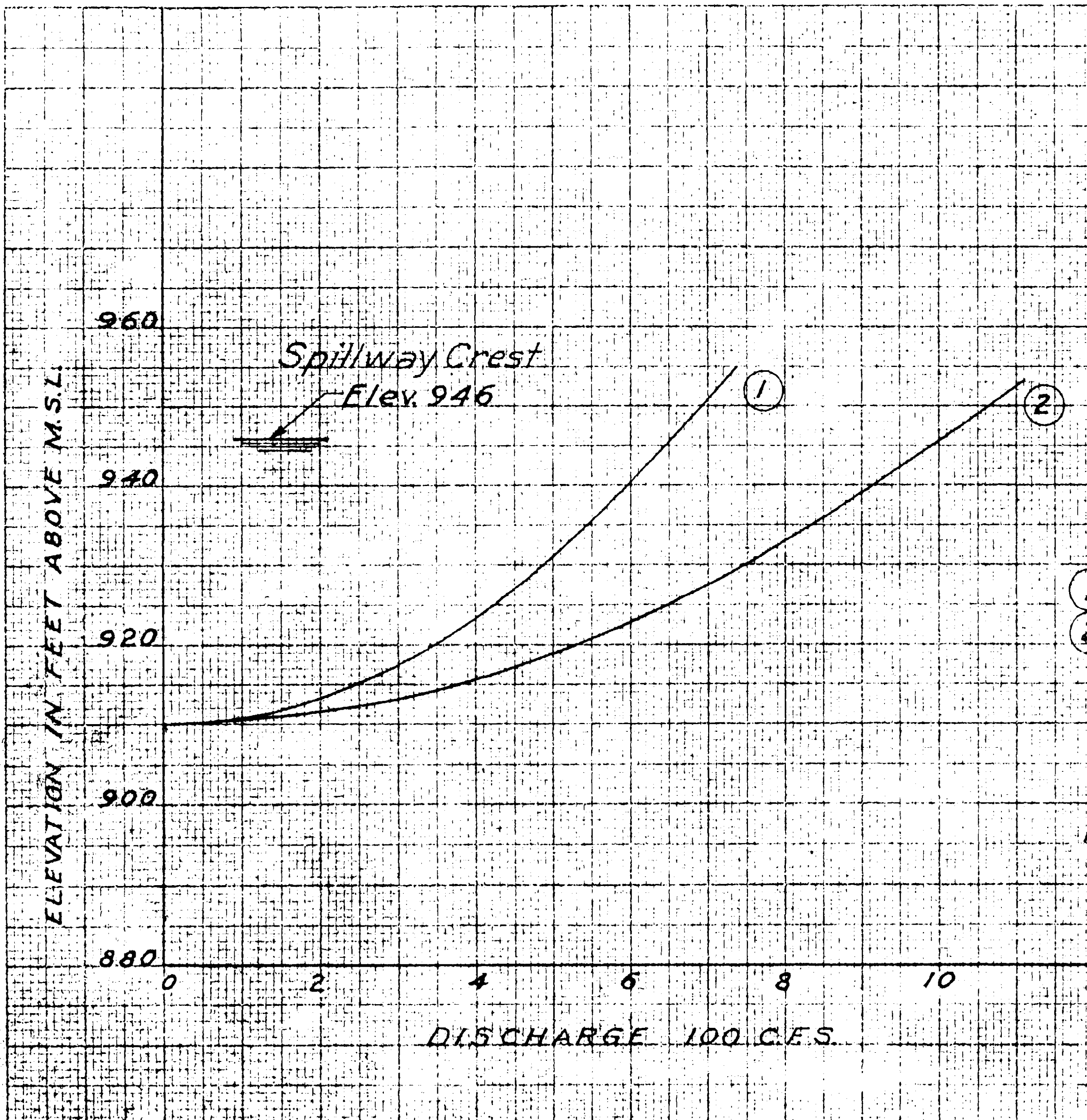

FLOOD DISCHARGES
NORTHEASTERN STATES
SHEET 1 OF 4 SHEETS
OFFICE CHIEF OF ENGINEERS, WASH., D.C.
SEPT., 1938
INDEX MAP
SHEET 1
SHEET 2
SHEET 3
SHEET 4
MOUNTAIN BROOK
WEST PETERBORO
BLACKWATER
W. HOPKINTON
HOPKINTON-EVERETT RESERVOIR
FRANKLIN FALLS
DISCHARGE IN C.F.S. PER SQ. MI.
- NOTES -
PEAK FLOWS WERE USED WHEN OBTAINABLE.
NUMBERS REFER TO ACCOMPANYING TABLE
OF DATA. SEE TABLE I. E.B.R.&H. NO. 11, 1938
- NOTE -
POINTS PLOTTED THUS: ⊕
ARE THE PEAK INFLOWS FOR
THE ADOPTED SPILLWAY DESIGN
FLOODS FOR THE RESERVOIRS DESIGNATED

PLATE 24



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
EFFECT ON 1936 FLOOD
U.S. ENGINEER OFFICE BOSTON, MASS.
APRIL, 1940

PLATE 2



MERRIMACK VALLEY FLOOD CONTROL

WEST PETERBORO RESERVOIR

OUTLET RATING CURVE

U.S. ENGINEER OFFICE BOSTON, MASS.
JULY 1940

PLATE 2

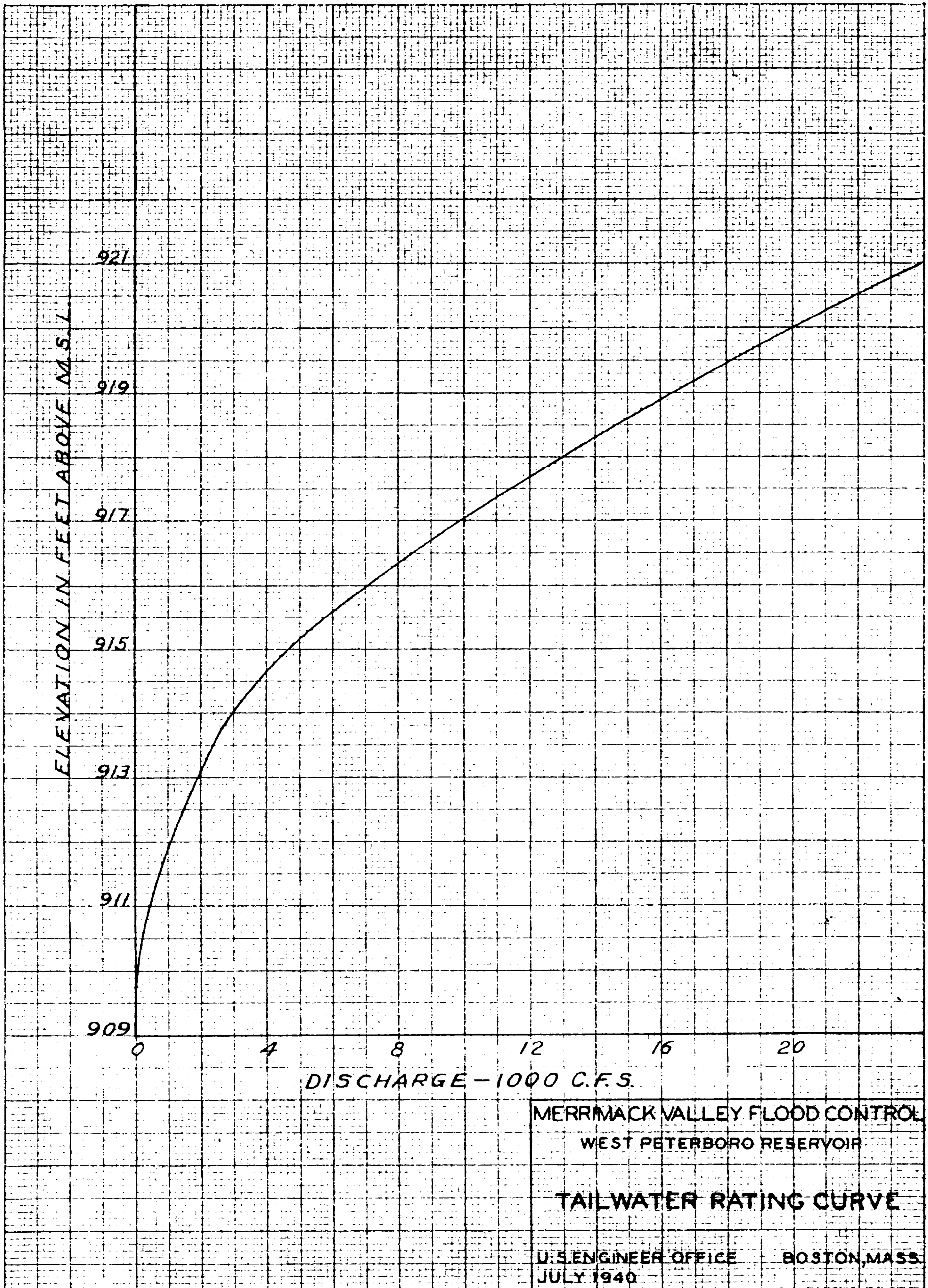
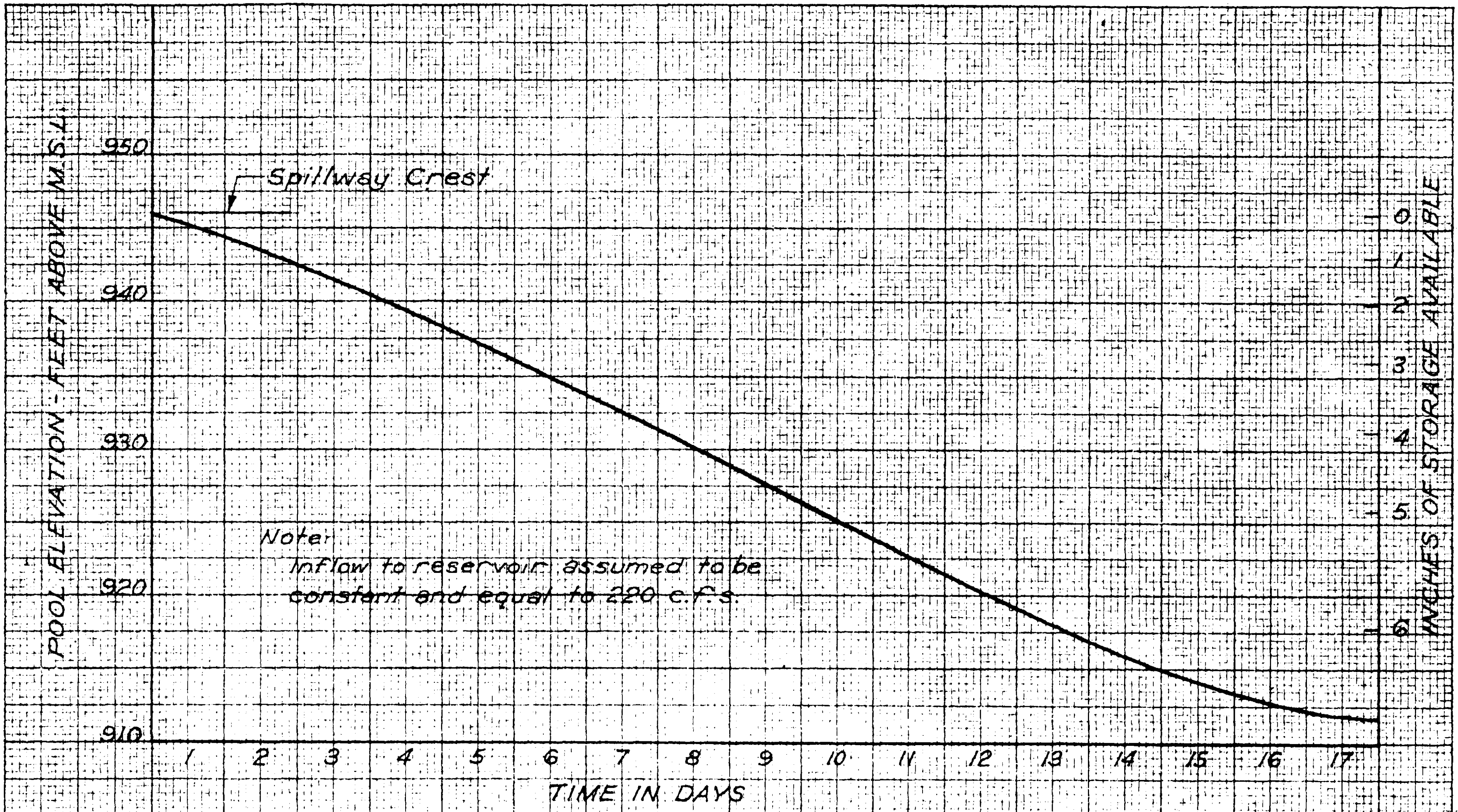

921
919
917
915
913
911
909
ELEVATION IN FEET ABOVE M.S.L.
0
4
8
12
16
20
DISCHARGE — 1000 C.F.S.
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
TAILWATER RATING CURVE
U.S. ENGINEER OFFICE
BOSTON, MASS.
JULY 1940

PLATE 27

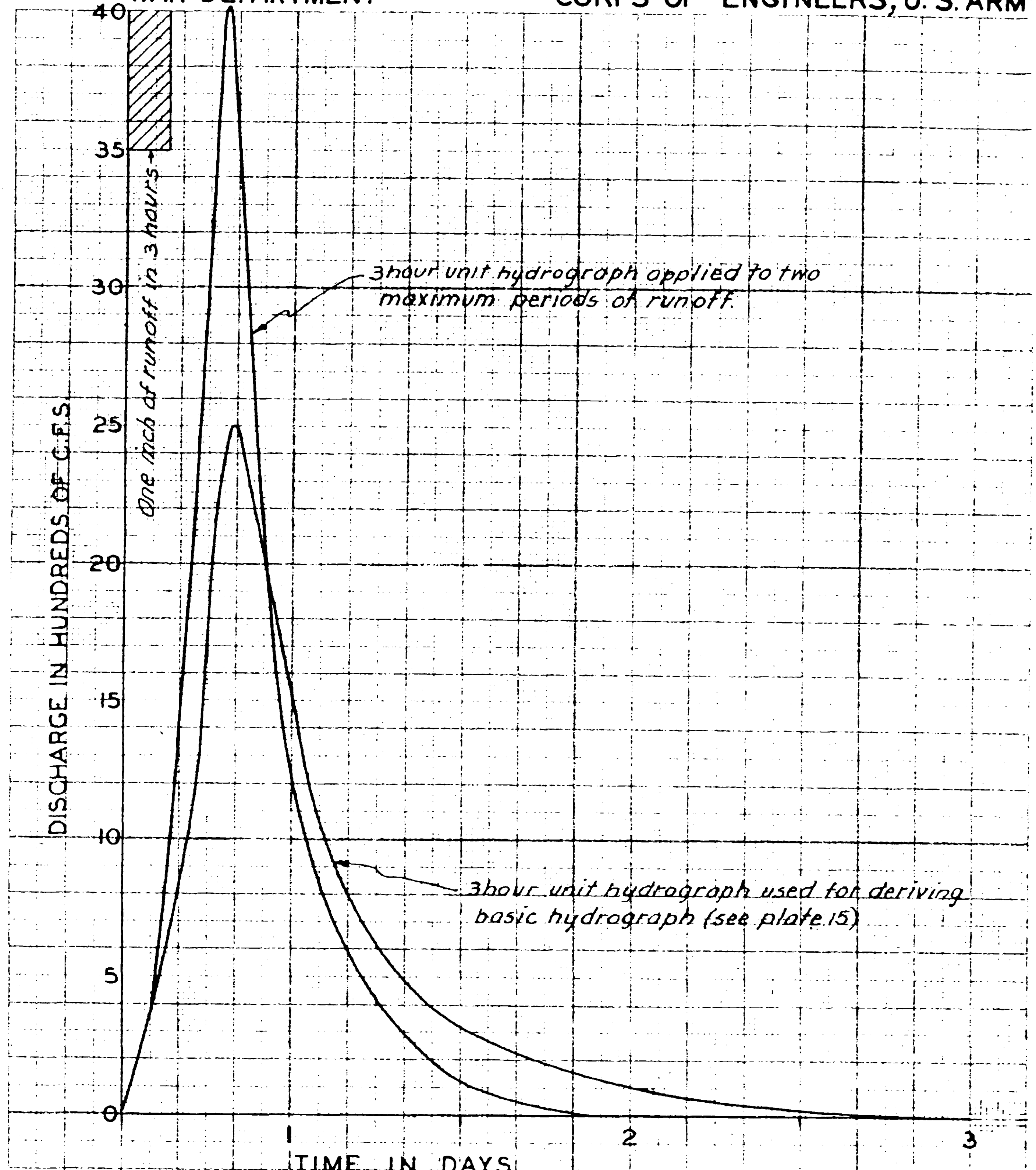


MERRIMACK VALLEY FLOOD CONTROL

WEST PETERBORO RESERVOIR

TIME TO EMPTY RESERVOIR

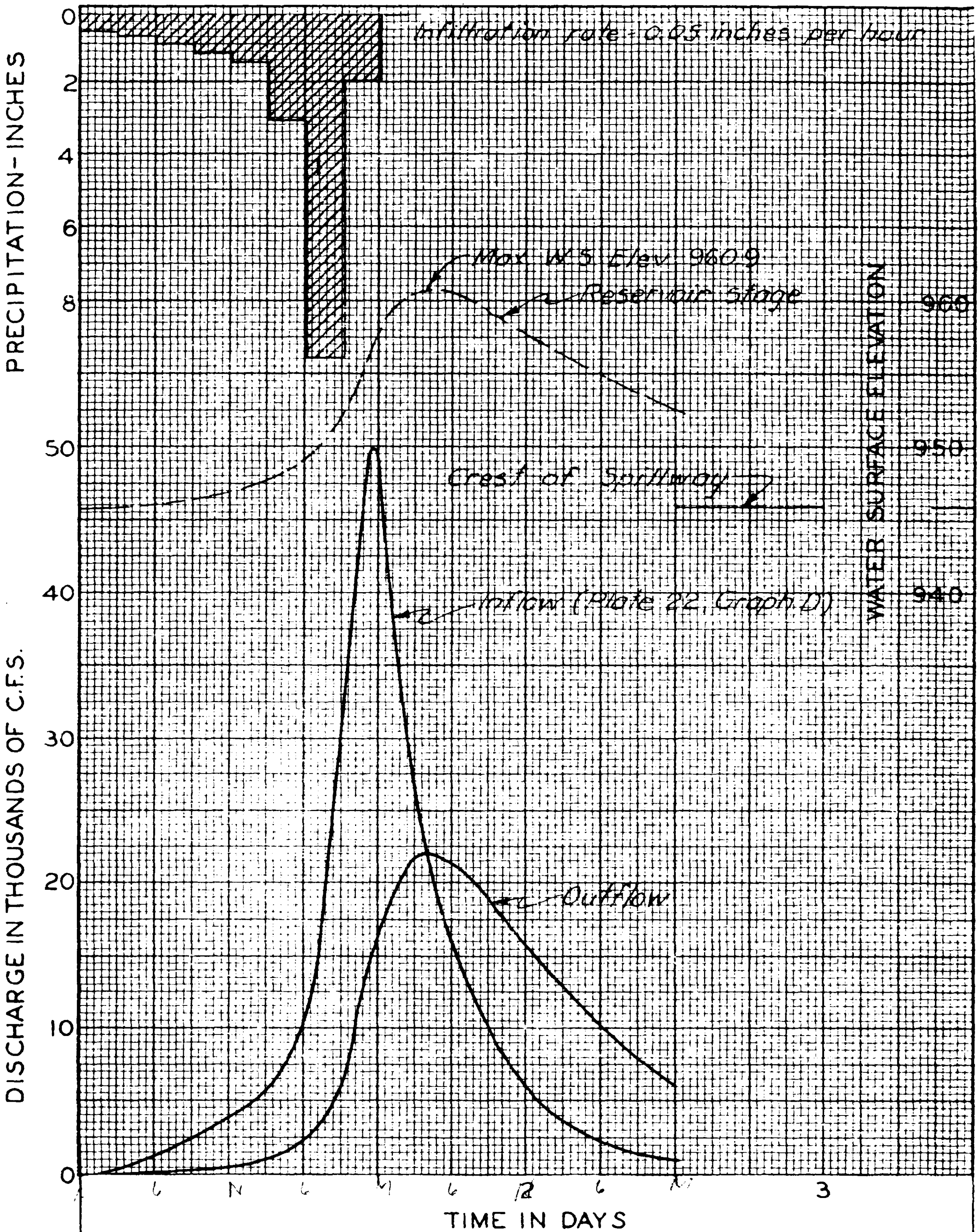
U.S. ENGINEER OFFICE BOSTON, MASS.
JULY 1940



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
SPILLWAY DESIGN FLOOD
INFLOW UNIT HYDROGRAPHS

U.S. ENGINEER OFFICE BOSTON, MASS.
DEC. 1940. FILE NO.

WAR DEPARTMENT　CORPS OF ENGINEERS U. S. ARMY



Note:-
Based on maximum Summer-Fall conditions

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

SPILLWAY DESIGN FLOOD
INFLOW - OUTFLOW
STAGE - HYDROGRAPH

U.S. ENGINEER OFFICE　BOSTON, MASS
DEC. 1940.　FILE NO.